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A BITUMINOUS CONCRETE ROAD ON ITS TWENTIETH BIRTHDAY

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G. P. St. CLAIR, *Editor*

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions.

In This Issue

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STATIC LOAD AND IMPACT TESTS OF LIGHTWEIGHT BRIDGE FLOOR SLABS

REPORT OF A COOPERATIVE INVESTIGATION CONDUCTED BY ALLEGHENY COUNTY, PA., AND THE UNITED STATES BUREAU OF PUBLIC ROADS

By L. W. TELLER, Senior Engineer of Tests, and G. W. DAVIS, Associate Engineer of Tests, United States Bureau of Public Roads

THE Division of Tests of the Bureau of Public Roads has recently completed a series of static load and impact tests on two bridge floor slabs of a rather unusual type. These tests were undertaken at the request of the Bureau of Public Works of Allegheny County, Pa., and were carried out as a cooperative project with that organization.

The slabs tested are a combination of steel shapes in the form of a grating or mat which, after erection, is filled with Portland cement concrete. The features of particular interest are the small depth, 3 inches; and the moderate weight, between 50 and 55 pounds per square foot. The advantages of a bridge floor slab having these characteristics are obvious.

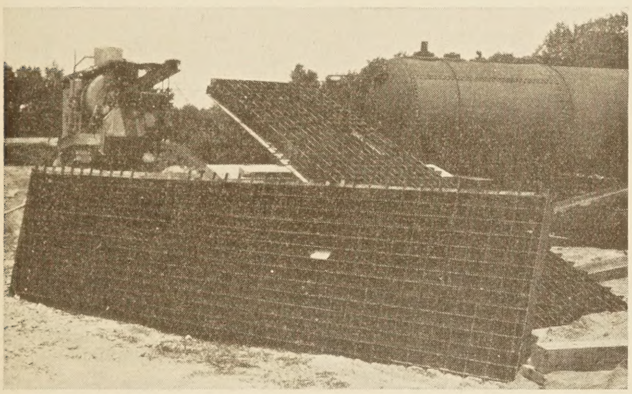


FIGURE 1.—APPEARANCE OF FACTORY-ASSEMBLED MATS FROM WHICH FIRST SLAB WAS CONSTRUCTED

Static load tests on small sections of a slab of this general type had indicated that such construction possesses flexural strength sufficient to warrant its consideration for use in the design of highway-bridge floors,[1] but there was some question as to whether or not the impact of the wheels of motor vehicles might break the bond between the steel and the concrete, thus weakening the slab structurally and also permitting the entrance of moisture between the two materials.

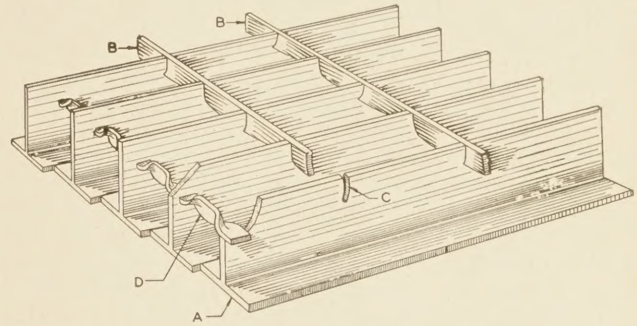
The primary object of the tests carried out by the Bureau of Public Roads was to determine to what extent severe motor-vehicle impact would affect the structural strength of bridge floor slabs of this type.

To obtain this information a definite schedule of static loads was applied to the slabs before, during, and after the program of impact loadings to which they were subjected. Deflection and strain measurements for each static loading made possible comparisons of the structural action of the slabs before and after receiving impact.[2]

DETAILED DESCRIPTION OF THE SLABS

While the two slabs tested were of the same general type and appearance, they differed from each other in a number of details and are therefore described separately. The slab built and tested in 1930 is referred to as the first slab and that built and tested in 1931 as the second slab.

The first slab was 12 feet, 7½ inches by 14 feet, 10½ inches in area and 3 inches thick. It was made up of five factory-assembled mats 35¾ inches wide and 12 feet, 7½ inches long. (See Fig. 1). Each of these mats was composed of 13 T-bars each with a 2½-inch base or flange and a 3-inch stem. These were assembled with their bases in contact, side by side, forming the bottom of the mat. The stems were locked together by means of 1 by ⅛ inch flat cross bars which were pressed into curved slots cut in the vertical stems of the T-bars. These cross bars were placed on 4-inch centers. Figure 2 shows the details of this construction. It will be noted that the slots in alternate T-bars curve in opposite directions. The cross bars are forced into these slots under heavy pressure and are caused to twist



A – BASE OF T-BARS   B – CROSS BARS BETWEEN STRINGERS
C – CURVED SLOTS   D – CROSS BARS ABOVE STRINGERS

FIGURE 2.—ASSEMBLY OF STEEL SECTION OF FIRST SLAB

in opposite directions at the successive junctions with the T-bars, securely locking the whole upper surface in place. As these slots are above the neutral axis of the T-bars, the lower section, which is subject to tension, is left intact and the compressive strength of the upper section is maintained presumably by the wedging action of the cross bars. In those sections of the mats above the supporting stringers, where negative moments are encountered, the position of four cross bars was changed, the slots being punched through the stems halfway between the upper and lower surface of the mat, as shown in Figure 2.

The steel mats were placed transversely on the stringers and were attached to them by spot welding through notches cut in the edge of the base of every other T-bar, to the center of the flange of the stringer.

The abutting edges of adjacent mats were bolted together at points midway between the stringers by five ½-inch bolts spaced 4 inches on center and passing through the stems of the T-bars which formed the edges of the mats. The result of this construction was a smooth grated surface with openings or pockets 2½ by 4 inches in size, the appearance of which, before being filled with concrete, is shown in Figure 3.

The concrete used in the first slab was of 1:1.5:3 proportions (by dry-rodded volumes). The water-cement ratio was 0.86. The aggregates used were Potomac River sand and a siliceous gravel from Fredericksburg,

[1] Engineering News-Record, vol. 104, No. 2, Jan. 9, 1930. Development tests on a light floor for bridges, by Leon S. Moisseiff.

[2] For a description of another investigation of the effect of static and impact loadings on bridge floor slabs of a different type see PUBLIC ROADS, vol. 8, No. 8, October, 1927, "Tests of the Delaware River Bridge Floor Slabs," by George W. Davis.

Va. The maximum size of the coarse aggregate was three-fourths inch.

The concrete was mixed in a power-driven mixer for two minutes, wheeled to the slab and vibrated into place with an electrically operated vibrator, as shown in Figure 3. After the concrete had been vibrated into place the surface was struck off with a wooden screed and roughly floated with a wooden float. It was then covered with wet burlap until the next morning, when the burlap was

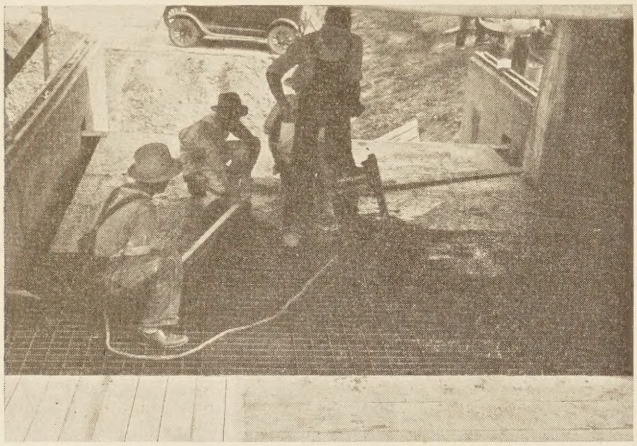


Figure 3.—Completed Steel Mat of First Slab Being Filled with Concrete by Means of Electric Vibrator

replaced by a 6-inch layer of straw. The straw was kept wet for 10 days, after which the slab was exposed to the air until the beginning of the tests at the age of 40 days.

The concrete, as it came from the mixer had an average slump of 3¼ inches. Five 6 by 12 inch compression cylinders made at this time and cured in a damp room showed an average compressive strength at 28 days of 3,360 pounds per square inch.

The second slab was 12 feet, 7½ inches by 15 feet, 4 inches in area and 3 inches thick. As with the first slab, it was made up of five factory assembled mats 12 feet, 7½ inches long and 36 inches wide. In this mat the T-bars were 3 inches wide on the base and twelve were used to a mat. The bases of adjacent T-bars were held together at four places (in the 12 foot, 7½ inch length) by electric welds 2 inches long.

The stems of the T-bars in the second slab were tied together at 4-inch intervals by transverse ⅝-inch, half-round bars, welded with the flat side up, into the upper edge of the stems. (See fig. 4). At the edges of the 3-foot mat sections, the ends of these bars were bent aside and downward so that when embedded in concrete they served to unite adjacent sections.

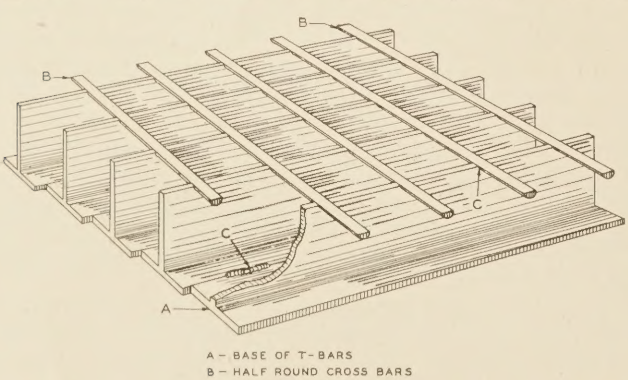


Figure 4.—Assembly of Steel Section of Second Slab

The appearance of the five sections of mat, when placed on the stringers, as in the case of the first slab, and welded, is shown in Figure 5.

The concrete which was used to fill the mat of the second slab was of the same proportions and aggregates as that used in the construction of the first slab. The

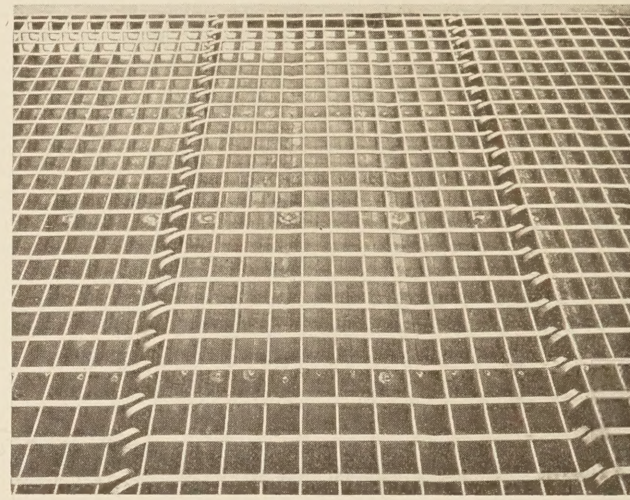


Figure 5.—Completed Mat Used in Construction of Second Slab Before Being Filled With Concrete. Note Number and Position of Welds and also Method Used for Uniting Edges of Individual Mats

cement, however, was of a different brand. A water-cement ratio of 0.82 was used, resulting in an average slump of 2¾ inches. Six test cylinders cured in a damp room showed an average compressive strength at 28 days of 5,030 pounds per square inch.

The mixing, placing, finishing, and curing operations for the concrete of the second slab were the same as those used in the construction of the first slab.

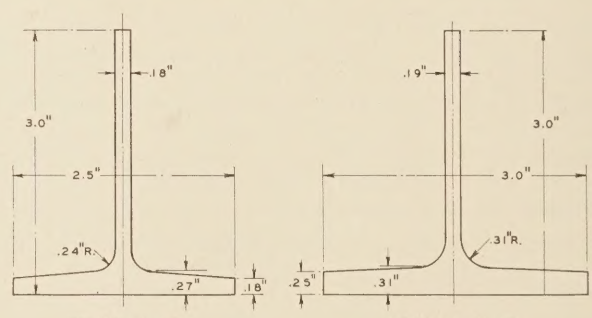


Figure 6.—Cross Sections of T-Bars Used in the Two Slabs

Figure 6 shows sections of the T-bars used in each slab. The calculated weight of the slabs is given in the following tabulation:

Weight per square foot of floor, in pounds

	First slab	Second slab
T-bars	18.0	20.0
Cross bars	1.3	1.6
Concrete	31.6	30.9
Total	50.9	52.5

In each test the slab was supported by four stringers set on 4-foot centers. These stringers were 18-inch 55-pound I-beams, rivet connected at their ends to

24-inch 80-pound **I**-beams. The floor beams were supported at their ends, in turn, by the concrete abutment walls, being held in their seats by anchor bolts. The distance between the floor beams was 16 feet and their span was 15 feet.

DESCRIPTION OF THE TEST PROGRAM

Both slabs were subjected to exactly the same program of tests. Since the object of the investigation was primarily to determine the effect of impact, the major part of the program was devoted to loadings intended to develop this information, and these loadings were applied at the center of the slab. Certain additional information was desired by Allegheny County, and the edge and 2-point loadings were included to provide these data. The detailed program follows:

1. Static loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied at—

   a. The center of the slab;

   b. A point 1 foot from the edge of the slab and midway between the two center stringers;

   c. Two points equidistant from the center of the slab along the axis transverse to the stringers, each point bearing half the load.

For all of the above loadings the deflections of both the slab and **I**-beams were determined and strains were measured both in the T-bars and in the two center stringers at the points shown on Figures 7 and 8.

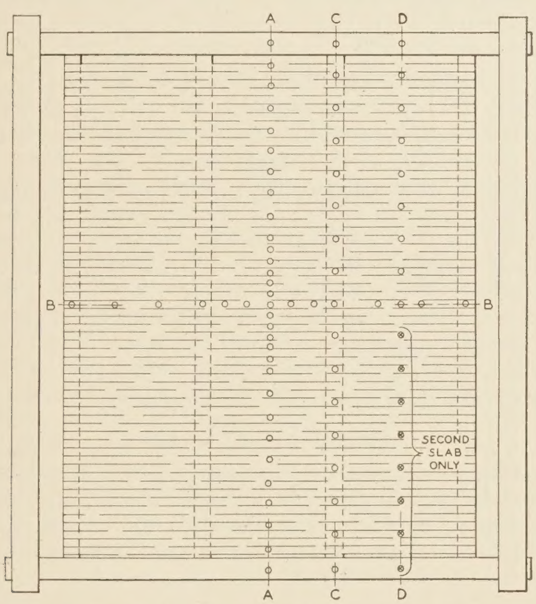


FIGURE 7.—PLAN OF SLAB SHOWING DEFLECTION POINTS

The spread or separation of the adjoining edges of the bases of the two T-bars directly under the load was measured for all of the loads applied at the center of the slab and for the 2-point loading.

2. A series of 1,000 impact blows simulating those delivered by the wheel of a heavy motor truck was applied at the center point of the slab.

3. Static loads as in 1, *a* were applied and the same measurements of deflection and strain were made.

4. A second series of 1,000 impact blows was applied at the center of the slab, the average maximum force developed being about 11 per cent greater than on the first series of impact.

5. Static loads of 5,000, 10,000, 15,000, 20,000, 30,000, and 40,000 pounds were applied at the center of the slab. Deflection and strain measurements were made as in the case of the previous static loadings.

6. A third series of impacts was applied at the center of the slab, the maximum force being very nearly the same as in the second impact series.

7. A fourth and last series of static loads was applied at the center of the slab, the load magnitudes, deflection, and strain measurements being the same as in 1, *a*.

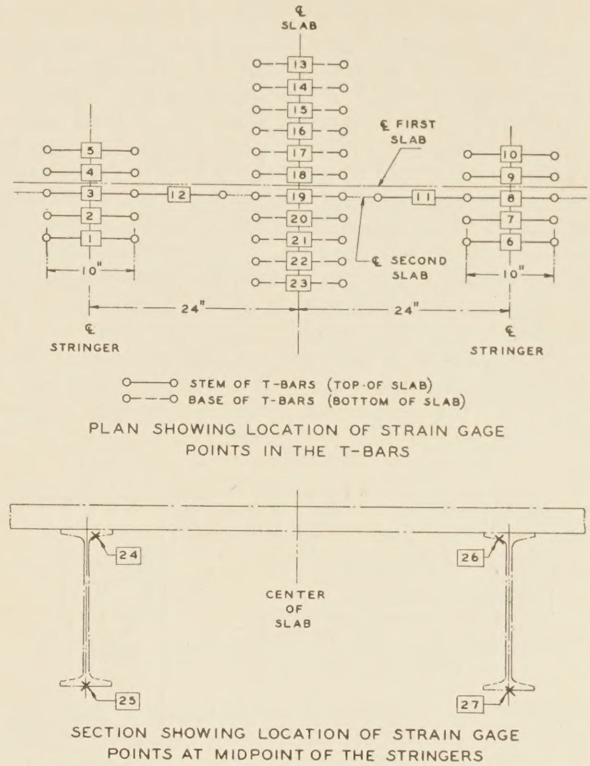


FIGURE 8.—LOCATIONS OF STRAIN-GAGE POINTS

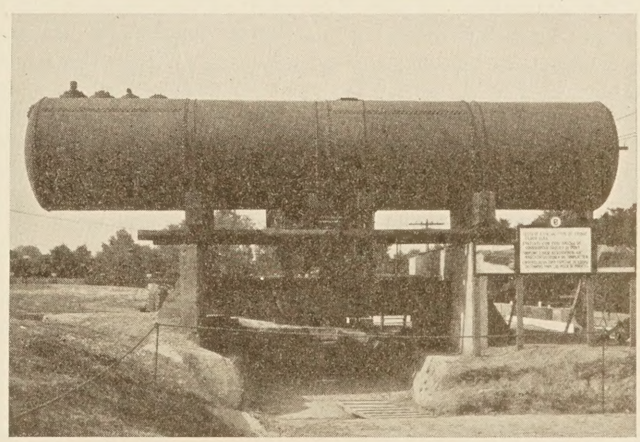


FIGURE 9.—GENERAL ARRANGEMENT OF TEST SLAB, SUPPORTING WALLS, AND TANK USED FOR THE APPLICATION OF THE STATIC LOADS

DESCRIPTION OF THE TESTING EQUIPMENT

Because of the large loads necessary for the static loading of these slabs and because of the alternate application of static and impact loads, a special set-up was designed to permit the execution of the program with facility.

The static loads were applied to the slabs with a hydraulic jack. This jack reacted against a large cylindrical steel tank containing water. The concrete walls, to which the ends of the floor beams were fastened, were extended 2 feet above the surface of the slab and supported the tank at an elevation sufficient to permit the installation of the jacking apparatus beneath it. On the tops of the two walls steel track was fastened, on which rested the rollers which supported the tank. The walls were made long enough to allow the tank to be moved back out of the way when impact tests were being made. The tank when filled with water weighed approximately 60,000 pounds. Figure 9 shows the general arrangement of the tank, slab, and supporting walls.

Attached to the under side of the tank was a beam of heat-treated steel which took the thrust of the jack. The load-deflection rate of this beam was known. To determine the magnitude of the loads being imposed on the slab, the deflection of the beam was measured with a micrometer dial. At the lower end of the jack a bearing block shod with segments of a solid rubber tire was provided. The details of this loading equipment are shown in Figure 10.

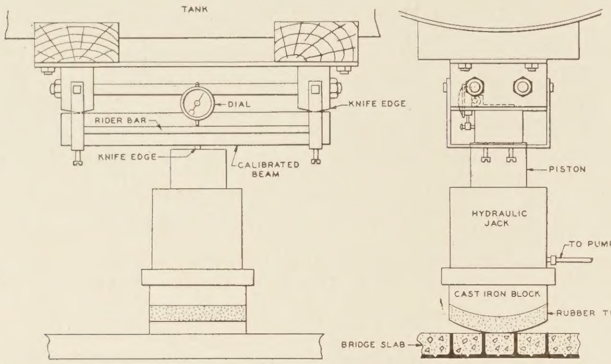


FIGURE 10.—APPARATUS USED IN APPLICATION AND MEASUREMENT OF LOAD

DEFLECTION MEASUREMENTS

The deflection of the slab and of the supporting I-beams was determined by measuring accurately changes in distance between gage points drilled into the lower surface of the slab or I-beams and corresponding gage points located directly beneath these in strips of steel fastened to concrete bases cast in the ground below the slab.

Changes in distance were measured with micrometer dials, reading in thousandths of an inch, fixed to the ends of wooden staffs. The staff and the dial were provided with conical steel points which fitted into the respective gage points.

Two pairs of gage points were set in the abutment walls to serve as a standard to check the length of the staffs at any time during a series of measurements.

The soil between the abutment walls was excavated to a lower level to give headroom under the slab for the measuring operations and to provide undisturbed earth as a foundation for the concrete bases carrying the deflection gage points. Figure 11 gives a good idea of the arrangements for the deflection measurements.

All of the deflection measurements were made with two movable staffs of different length, one being used under the stringers and the other under the T-bars of the slab proper. These staffs were moved from point to point after the desired load was on the slab. The deflection of the floor beams was measured by similar staffs which, however, were left in position and not moved from point to point. The positions of all deflection points are shown in Figure 7.

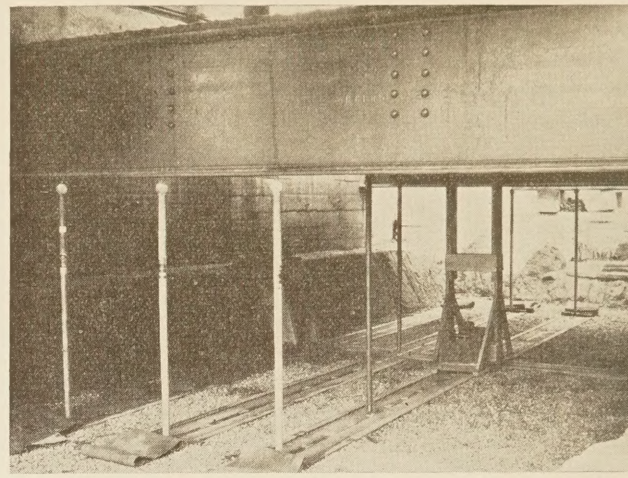


FIGURE 11.—LOWER DEFLECTION POINTS AND MICROMETER DIALS USED FOR THE MEASUREMENTS

STRAIN MEASUREMENTS

In these tests the strain measurements were confined to the steel and the strain gage positions were located in the regions where relatively large moments would be expected under the center loading. Figure 8 shows the locations at which strains were measured.

For making the measurements a 10-inch Whittemore strain gage was used. This gage was developed in connection with the Arch Dam Investigation conducted under the direction of the Engineering Foundation, and has been described in detail elsewhere.[3]

Briefly, it consists of parallel side bars of nickel steel connected at their ends by steel spring fulcrum plates which maintain the alignment of the bars. Two pointed legs for insertion in the gage holes are provided at opposite ends of the gage, one leg being attached to each side bar. The relative displacement of the side bars longitudinally is indicated by a micrometer dial reading directly in ten-thousandths of an inch. Corrections for temperature changes are made by reference to an unstressed steel bar, in the usual way.

Strain gage holes were drilled with a No. 56 twist drill and the burr removed with a special reamer.

SPREAD OF THE T-BARS

In order to determine the amount of separation that occurred between adjacent T-bars directly beneath the loaded area, a small micrometer dial was attached to the bottom of the base of one T-bar and the stem of this dial acted against a small stud in the adjacent T-bar.

IMPACT TESTS

Impact was applied by means of the portable impact machine developed by the Bureau of Public Roads and described in PUBLIC ROADS several years ago.[4] The method of test was the same as that used in the tests of the Delaware River Bridge floor slabs, referred to above.

In these tests the acceleration of the unsprung mass was measured by means of an accelerometer of the single-element contact type.[5]

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[3] Instruments, vol. 1, No. 6, p. 299, June, 1928, Whittemore Strain Gage, by H. L. Whittemore.

[4] PUBLIC ROADS, vol. 5, No. 2, April, 1924, Impact Tests on Concrete Pavement Slabs, by Leslie W. Teller.

[5] For discussions of the use of accelerometers for the measurement of motor vehicle impact see the following publications: (1) Proceedings, American Society for Testing Materials, 1923, An Accelerometer for Measuring Impact, by E. B. Smith; (2) PUBLIC ROADS, vol. 5, No. 10, December, 1924, Accurate Accelerometers Developed by the Bureau of Public Roads, by L. W. Teller; (3) PUBLIC ROADS, vol. 11, No. 5, July, 1930, Calibration of Accelerometers for Use in Motor Truck Impact Tests, by J. A. Buchanan and G. P. St. Clair.

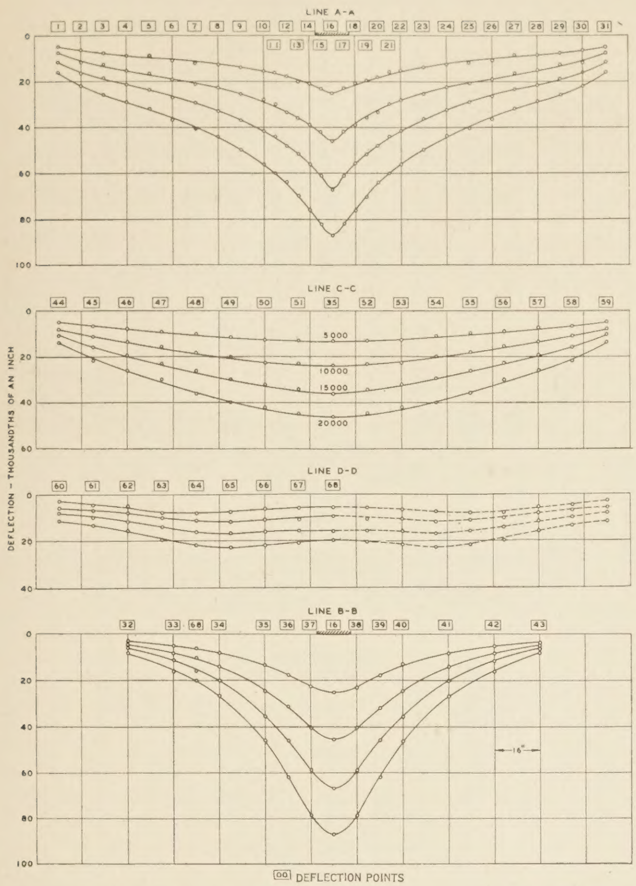

FIGURE 12.—DEFLECTIONS OF FIRST SLAB UNDER CENTER LOADING BEFORE IMPACT

DETAILED DESCRIPTION OF THE TESTING OF THE FIRST SLAB

Static loads at the center.—Before any deflection or strain data were taken, several preliminary static loads were applied to the center of the slab. The magnitude of each successive preliminary load was increased until a load of 20,000 pounds had been applied. The object of these loadings was to place the structure in a cyclic state or, in other words, in a condition such that successive applications of a given load would produce the same structural effects. These loadings were also utilized for necessary adjustments of the loading and measuring apparatus.

Initial static loading.—Following the application of the preliminary loads, two series of initial test loadings were applied to the center of the slab. Each series consisted of loads of 5,000, 10,000, 15,000, and 20,000 pounds, and the two series were applied on successive days with a recovery period of about 18 hours between. Under each load of these two initial series complete deflection and strain measurements were made.

The zero readings for deflection and strain—i. e., measurements for the unloaded condition—were made with the entire loading apparatus lifted clear of the slab and in applying all of the static loads due allowance was made for the dead weight of all of the loading equipment below the calibrated beam.

In order to establish beyond question the data which were to serve as a basis of comparison for the stiffness of the slab before impact—i. e., the deflections and strains under the center loading—the third series of initial loadings were applied several days after the first two. The data obtained at this time consisted of one set of measurements for deflection at all of the deflection points on the slab for loads of 5,000 and 20,000

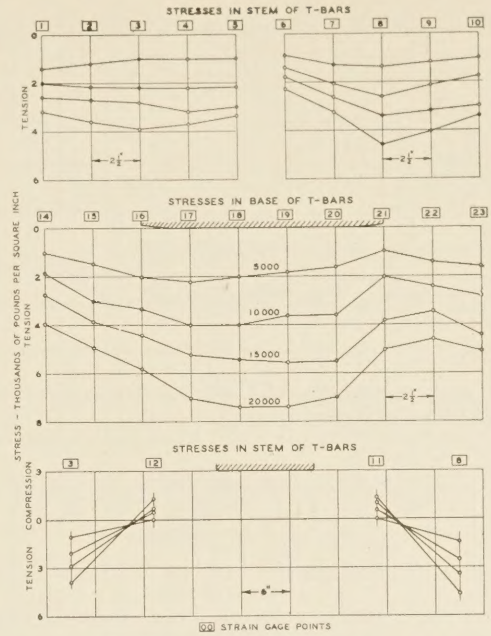

FIGURE 13.—STRESSES IN FIRST SLAB CAUSED BY STATIC LOADS BEFORE IMPACT

pounds and a series of measurements of deflection at deflection point 16 and strain measurements at strain gage point 15 for 10 applications each of 5,000, 10,000, 15,000, and 20,000 pound loads, made at approximately half-hour intervals.

The data obtained under successive applications of a given load agreed within very close limits and the mean values of all of the measurements made under each of the several loads were taken to express the load-deflection and load-strain conditions in the slab before impact was applied. These mean deflection curves are shown in Figure 12 and the strain data for the T-bars are shown in Figure 13 as stresses, computed with an assumed value for the modulus of elasticity of 30,000,000 pounds per square inch.

As the load was applied at the center of the slab, the deflection of points symmetrically spaced with respect to its axes should be equal. Actually this was found to be true, the differences in the measured deflections of symmetrically placed points being negligible. Therefore, throughout this report, in plotting the deflection curves for the various loads and lines of points the measured deflections of symmetrical points have been averaged.

With the load at the center point the separation or spread which occurred between the adjacent edges of the bases of the two T-bars directly underneath the load was measured, as previously described. These measurements were made for loads 5,000, 10,000, 15,000, and 20,000 pounds on each of two successive days. The two sets of values agreed very closely and are averaged in the following tabulation:

Load in pounds	Spread in inches
5, 000	0. 0018
10, 000	. 0035
15, 000	. 0050
20, 000	. 0064

Static loads at the edge.—Static loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied at a point 1 foot from the edge of the slab on the axis of the slab parallel to the stringers.

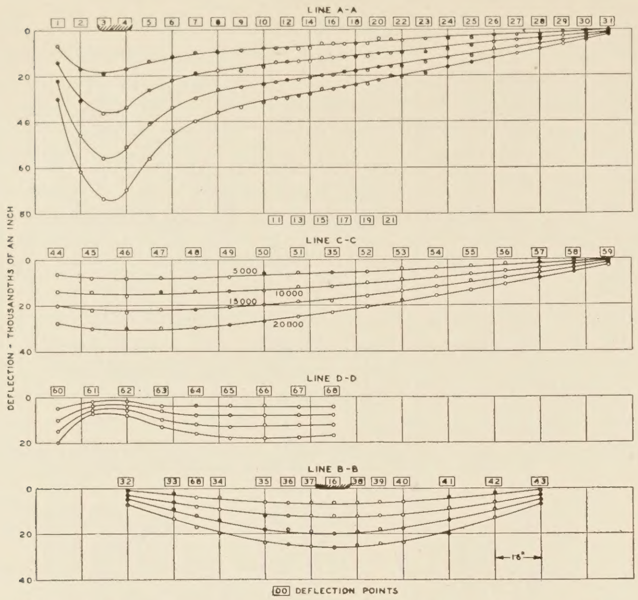

FIGURE 14.—DEFLECTIONS OF FIRST SLAB UNDER EDGE LOADING BEFORE IMPACT

The deflection curves for the slab and for the stringers nearest the load are shown in Figure 14. Two sets of measurements of deflection were made for two applications of each of the four loads on the same day. The two sets of values agreed with each other within close limits and the values shown are for the means of the two measurements.

No strain measurements of any significance were obtained under this loading since the strain gage locations were in areas of small stress. The indicated stress at the midpoint of the two center stringers was 3,300 pounds per square inch in the bottom flange under the 20,000 pound load.

The 2-point static load.—Another special loading was that applied equally on two tire segments set equidistant from the midpoint of the slab along the axis normal to the stringers. These tire segments were 72 inches apart, center to center.

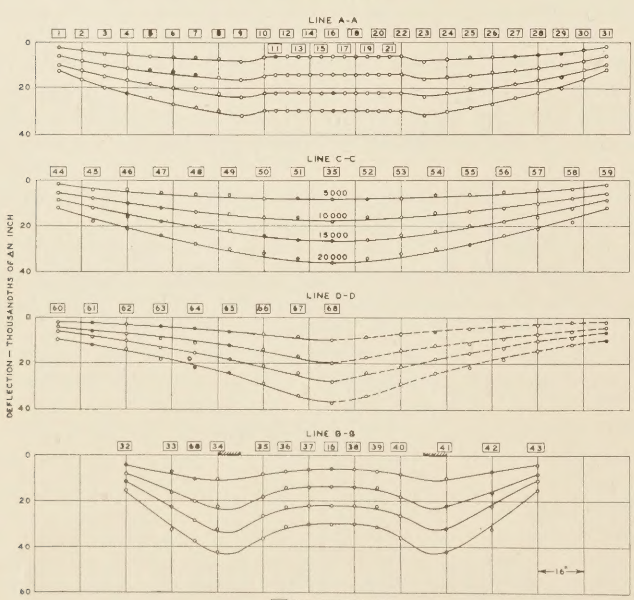

FIGURE 15.—DEFLECTIONS OF FIRST SLAB UNDER TWO-POINT LOADING BEFORE IMPACT

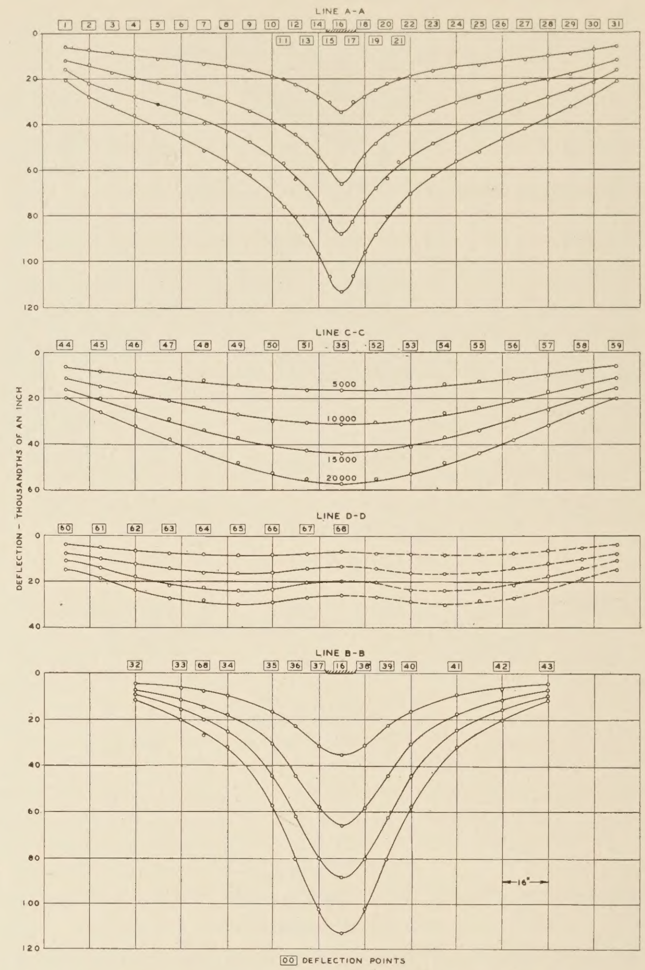

FIGURE 16.—DEFLECTIONS OF FIRST SLAB UNDER CENTER LOADING AFTER FIRST IMPACT

The deflection curves for loads of 5,000, 10,000, 15,000, and 20,000 pounds applied in this manner are shown in Figure 15. As in the case of the edge loading, the curves represent mean values from two sets of measurements. The strains produced at the strain gage locations shown in Figure 8 were too small to be determined with the strain gage used in these tests except at the midpoint of the lower flange of the center stringers where a maximum stress of about 3,000 pounds per square inch was indicated.

The spread of the bases of the two T-bars at the midpoint of the slab was also measured under the 2-point loading and was found to be very small, as shown in the following tabulation:

Total load, in pounds	Spread, in inches
5, 000	0. 0002
10, 000	. 0004
15, 000	. 0006
20, 000	. 0009

Application of the first impact.—In all of the impacts applied to the bridge floor slabs in these tests, the conditions selected were such that very severe impact was produced. The conditions approximated those of a loaded 5-ton truck, equipped with solid tires, the unsprung weight being 1,967 pounds and the spring deflection being that which would correspond to a sprung load of about 6,000 pounds. The tire used on

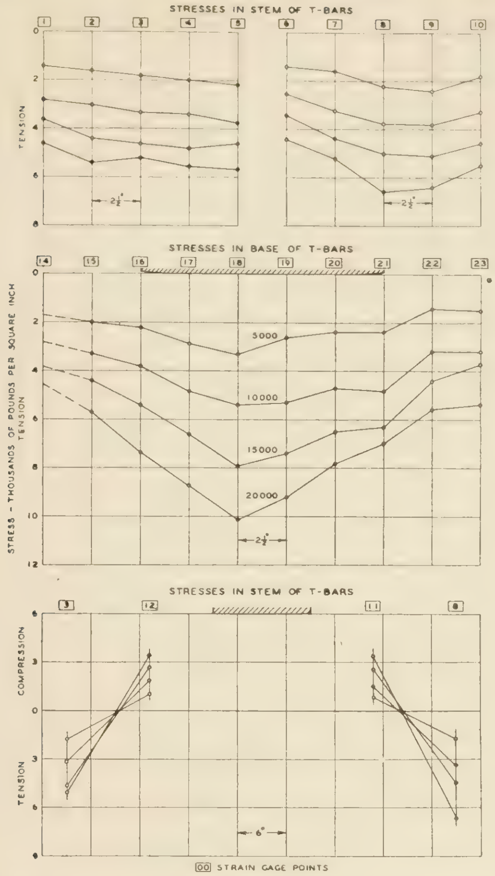

FIGURE 17.—STRESSES IN FIRST SLAB CAUSED BY STATIC LOADS AFTER FIRST IMPACT

the wheel of the impact machine in the tests of the first slab was a 36 by 6-inch solid rubber truck tire. With these conditions the first series of 1,000 drops of the impact machine were applied. A free fall of 0.25 inch was used. The measured accelerations varied between 310 and 354 feet per second per second, the average impact force for this series being 28,200 pounds. The rate at which the impact was applied was approximately seven blows per minute.

Careful observation of the slab during the application of the first impact and examination afterwards showed no visible effect of the severe hammering. The thin grout or laitance between the edges of the T-bars on the bottom of the slab remained in place even directly under the point of impact and no visible structural cracks appeared on the upper surface of the slab.

After the completion of these tests the impact machine was removed from the slab, the static loading equipment was replaced, and a second series of static loads applied at the center of the slab in accordance with the program.

Static loading after the first impact.—Loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied at the center of the slab and complete deflection and strain measurements were made for each load. Two sets of tests were made and the averaged data for the deflections are shown in Figure 16 and for the strains in Figure 17.

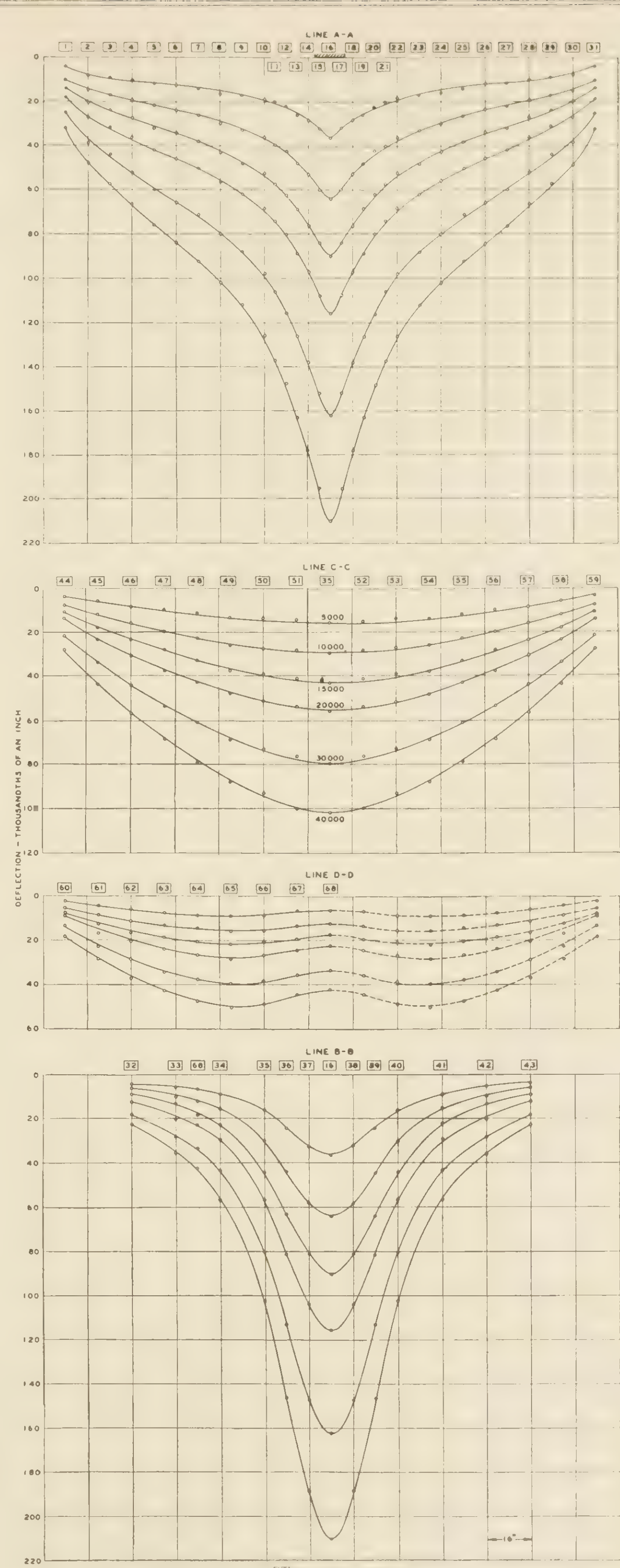

FIGURE 18.—DEFLECTIONS OF FIRST SLAB UNDER CENTER LOADING AFTER SECOND IMPACT

If these data are compared with the corresponding data obtained before the impact was applied it will be noted that for a given load there was a definite increase in the magnitudes of the deflections and of the stresses in the T-bars after the first series of impacts. For example, under a 20,000-pound static load at deflection point 16 (the center of the slab) an increase of from 0.087 to 0.113 inch (30 per cent) occurred. Similarly, the maximum stresses in the T-bars increased after impact from 4,650 to 6,600 pounds per square inch (42 per cent) over the stringers and from 7,350 to 10,200 pounds per square inch (39 per cent) directly underneath the load. Strain measurements in the top of the T-bars between the load and the stringers showed an increase in compressive stress of from 1,350 to 3,450 pounds per square inch (about 155 per cent) after the impact.

Application of the second impact.—The second series of 1,000 drops of the wheel of the impact machine was applied at the same point and in the same manner as the first series, except that the free fall of the wheel before the tire made contact with the slab was increased from 0.25 to 0.63 inch in order to produce an impact force of greater magnitude. The effect of this was to increase the range of measured accelerations from 310–354 to 378–426 feet per second per second. The average maximum impact reaction for the second series of 1,000 drops was 31,300 pounds.

This impact produced no visible change in the appearance of either surface of the slab.

Static loading after the second impact.—Upon the completion of the second series of impact tests, the machine was removed and static loads of 5,000, 10,000, 15,000, 20,000, 30,000, and 40,000 pounds were applied at the center of the slab. These loads were repeated, complete deflection and strain measurements being made for each loading. The deflection data are shown in Figure 18. The corresponding strain data, expressed as unit stresses, for all of the strain gage positions shown in Figure 8 are given in Table 1.

TABLE 1.—*Unit stresses in T-bars of first slab under center loading after second impact, in pounds per square inch*

TOP OF STEMS OF T-BARS

Load	Stress at strain gage point—											
	1	2	3	4	5	6	7	8	9	10	11	12
	Tension										Compression	
Pounds												
5,000	1,650	----	2,400	2,100	2,250	1,350	1,800	1,950	2,400	1,950	750	1,050
10,000	2,700	----	3,600	3,450	3,750	2,550	3,300	3,450	4,200	3,750	2,100	2,250
15,000	3,450	----	4,800	4,500	4,800	3,450	4,800	5,400	5,700	4,950	3,150	2,850
20,000	4,500	----	5,700	5,550	6,150	3,900	5,550	6,300	6,600	6,000	4,350	3,750
30,000	6,300	----	8,100	8,100	8,700	5,700	7,800	8,700	9,600	8,700	5,400	5,400
40,000	8,250	----	10,800	10,500	10,500	7,500	9,300	11,550	10,950	10,650	6,900	6,150

BOTTOM OF BASES OF T-BARS

Load	Stress at strain gage point—									
	14	15	16	17	18	19	20	21	22	23
	Tension									
Pounds										
5,000	------	1,050	1,650	1,950	2,700	2,850	2,100	2,400	1,800	900
10,000	------	2,700	3,450	4,200	5,700	5,250	4,500	4,050	2,850	1,650
15,000	------	4,200	5,100	5,700	8,100	7,050	5,850	5,100	4,500	3,300
20,000	------	5,850	6,600	7,950	10,200	9,450	7,800	7,050	5,400	4,050
30,000	------	8,400	9,600	11,700	15,000	14,100	12,000	9,900	8,400	6,300
40,000	------	11,100	13,200	15,300	19,800	18,450	15,000	12,150	10,650	9,000

A comparison of the data for the four lowest loads with those obtained under the same loads before the second impact show that there was no appreciable increase in either deflections or stresses. The data obtained for the 30,000 and 40,000 pound loads indicate that the action of the slab under these loads was elastic and no apparent damage to the slab was observed.

Application of the third impact.—The third impact consisted of a series of 1,000 drops, duplicating as closely as possible the second series. The same free fall was used and the average impact reaction for this series was 31,900 pounds.

Static loading after the third impact.—The final static tests consisted of the application of loads of 5,000,

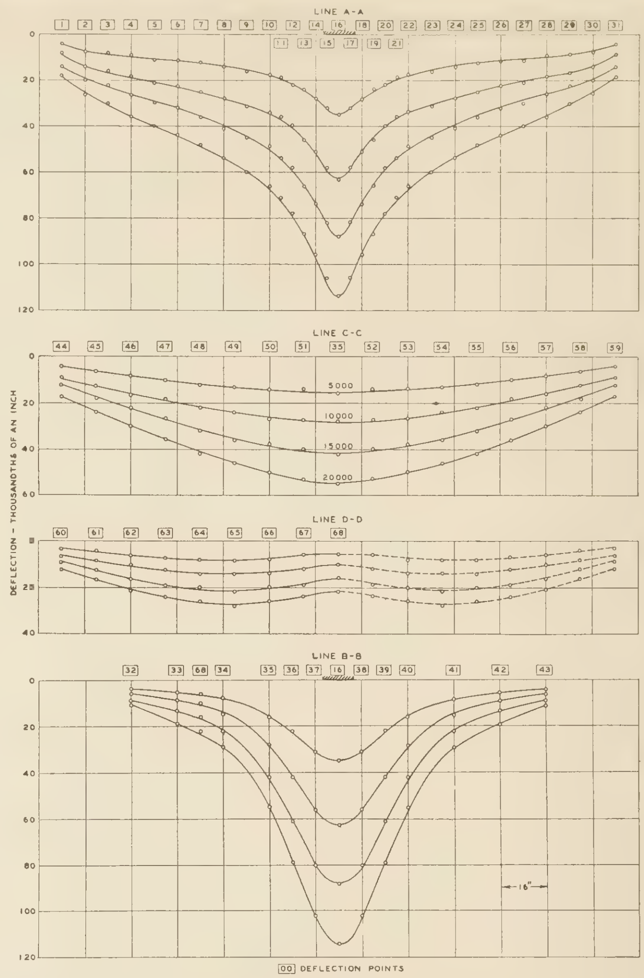

FIGURE 19.—DEFLECTIONS OF FIRST SLAB UNDER CENTER LOADING AFTER THIRD IMPACT

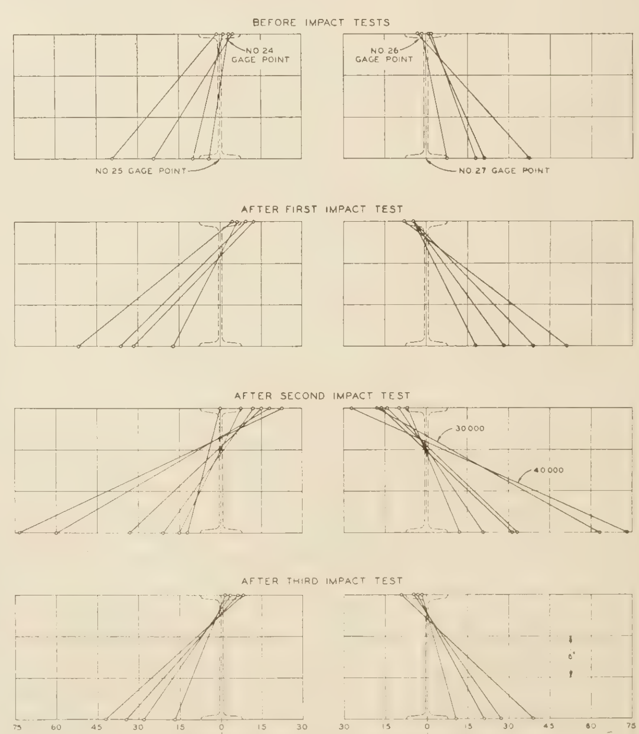

FIGURE 20.—STRESSES IN FIRST SLAB AT MIDPOINT OF STRINGERS UNDER STATIC LOADS OF 5,000, 10,000, 15,000, AND 20,000 POUNDS

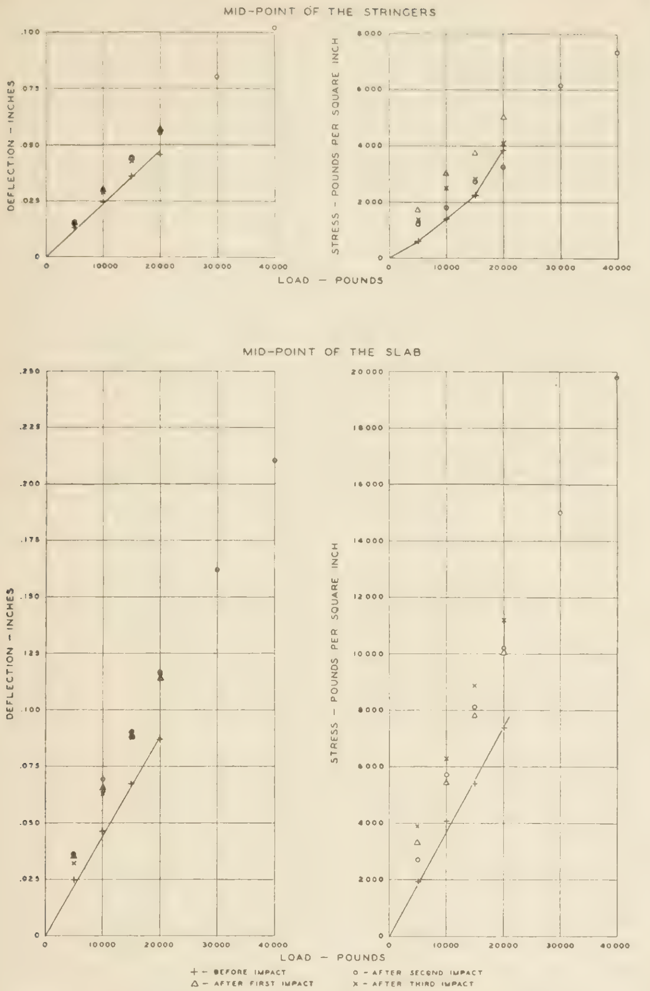

FIGURE 21.—COMPARISON OF DEFLECTIONS AND STRESSES IN FIRST SLAB BEFORE AND AFTER IMPACT

10,000, 15,000, and 20,000 pounds at the center of the slab with complete deflection and strain measurements for each loading. The deflection data for these loadings are shown in Figure 19 and the stress data in Table 2.

When these data are compared with those obtained for the same loads before the application of the third impact, it is evident that there is no significant difference.

Stresses in the stringers.—Mention has been made of the fact that strain measurements were made at the mid-points of the two stringers nearest the center line of the slab. The strains measured were those in the upper and lower flanges of the I-beams at the positions shown in Figure 8.

The strain data obtained at these points for all of the static loadings applied to the slab are shown in Figure 20 expressed as unit stresses. It is indicated that the stress in the upper flanges was extremely small and that the stress in the lower flanges did not exceed 5,100 pounds per square inch (except for the 30,000 and 40,000 pound loads). The position of the neutral axis of the stringers was not very definitely determined because of the small deformations measured in the upper flanges of the I-beams, but generally it appears to be from 12 to 15 inches above the lower flange.

In Figure 21 a comparison is made of the structural action of the stringers before and after impact and of the T-bars at the center of the slab before and after impact. For this purpose the load-deflection and the load-stress relations for the mid-point of the stringers

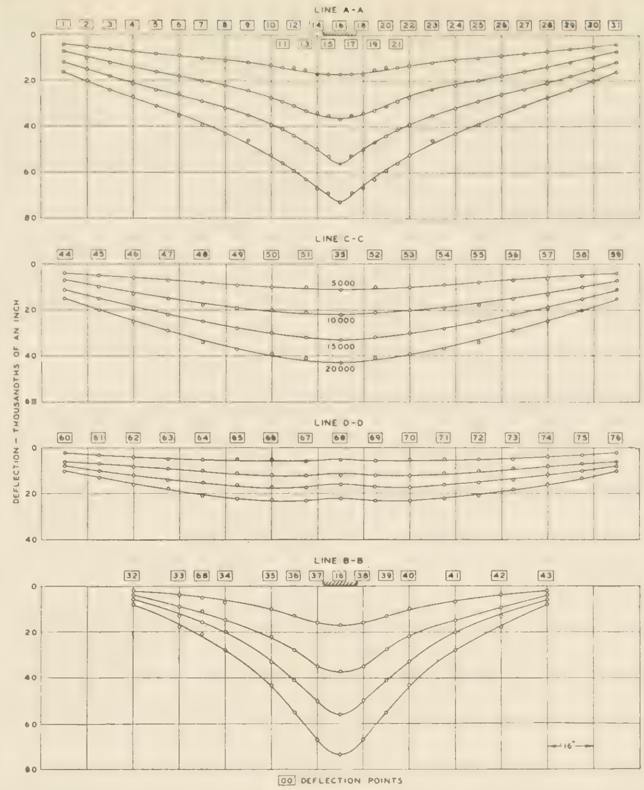

FIGURE 22.—DEFLECTIONS OF SECOND SLAB UNDER CENTER LOADING BEFORE IMPACT

TABLE 2.—*Unit stresses in T-bars of first slab under center loading after third impact, in pounds per square inch*

TOP OF STEMS OF T-BARS

Load	Stress at strain gage point—											
	1	2	3	4	5	6	7	8	9	10	11	12
	Tension										Compression	
Pounds												
5,000	1,950	----	2,400	2,250	2,250	1,350	1,950	2,250	2,550	2,400	750	1,050
10,000	3,300	----	3,750	3,600	4,200	2,700	3,450	4,200	4,050	3,750	1,800	2,250
15,000	4,200	----	5,100	5,100	5,100	3,900	4,950	6,000	5,700	5,250	2,550	3,150
20,000	4,950	----	6,300	6,150	6,300	6,000	6,150	7,200	6,900	6,600	3,750	4,050

BOTTOM OF BASES OF T-BARS

Load	Stress at strain gage point—									
	14	15	16	17	18	19	20	21	22	23
	Tension									
Pounds										
5,000	------	1,350	2,550	2,700	3,900	3,000	2,550	2,250	1,500	1,650
10,000	------	3,300	3,600	4,650	6,300	5,850	4,950	4,650	3,000	2,400
15,000	------	4,650	5,700	6,450	8,850	7,650	6,750	6,150	4,050	4,200
20,000	------	5,700	7,350	8,250	11,250	10,050	8,400	7,350	5,850	5,250

and for the base of the T-bar at the mid-point of the slab were selected. On this graph the values obtained before impact are connected with a solid line and the observed values after the several impact tests are simply plotted points.

DETAILED DESCRIPTION OF THE TESTING OF THE SECOND SLAB

Static loads at the center.—In testing the second slab the general procedure was exactly the same as that followed in the testing of the first slab. The same loading program was followed throughout except for slight variations in the magnitude of the impact forces applied.

Following a series of preliminary loadings, as in the case of the first slab, three series of initial static loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied at the center of the slab on three different days and complete deflection and strain measurements were made for each. The mean deflection curves obtained are shown in Figure 22 and the strain data expressed as unit stresses are shown in Figure 23.

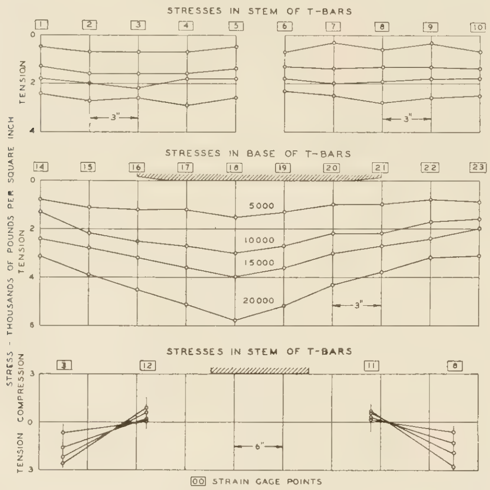

FIGURE 23.—STRESSES IN SECOND SLAB CAUSED BY STATIC LOADS BEFORE IMPACT

The spread between the bases of the two adjacent T-bars directly under the load was measured in the same manner as was used in the previous test and found to be very small, being only 0.003 inch for the 20,000-pound load.

Static loads at the edge.—Static loads of 5,000, 10,000, 15,000, and 20,000 pounds were applied twice at a point 1 foot from the edge of the slab and on the axis of the slab parallel to the stringers.

The deflection curves in Figure 24 are the mean values obtained in these loadings. The only significant strain measurements obtained were at the centers of

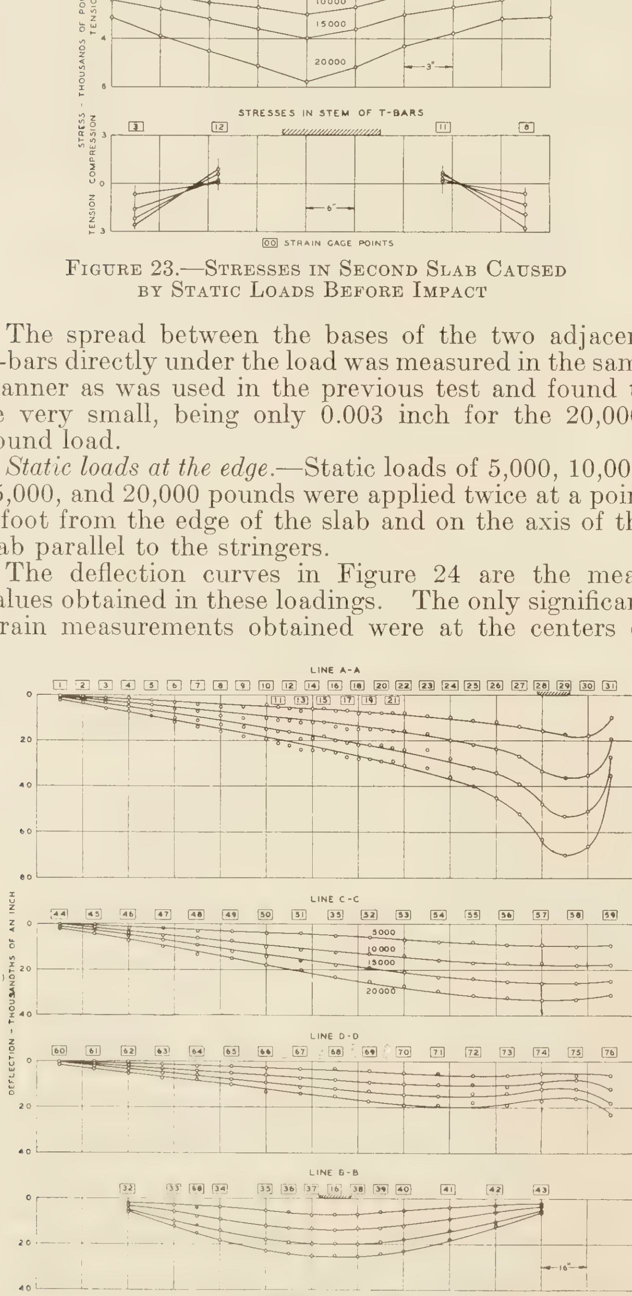

FIGURE 24.—DEFLECTIONS OF SECOND SLAB UNDER EDGE LOADING BEFORE IMPACT

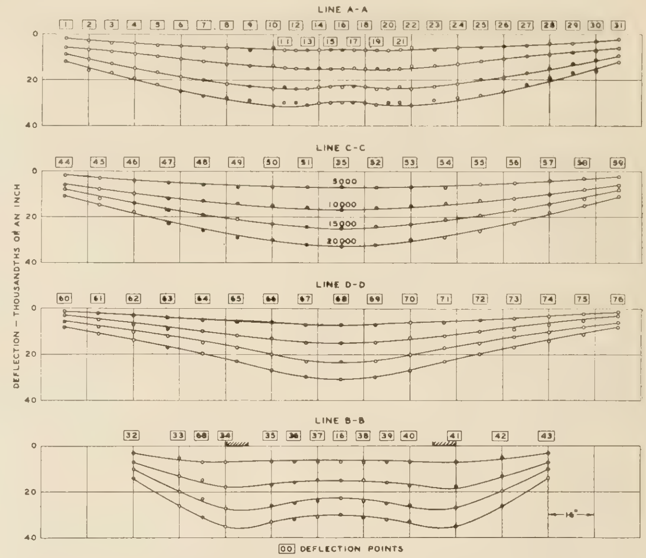

FIGURE 25.—DEFLECTIONS OF SECOND SLAB UNDER TWO-POINT LOADING BEFORE IMPACT

the stringers where stresses of about 1,000 pounds per square inch were indicated.

The reversal of the arrangement of the deflection curves for this loading in the presentation of the data for the two slabs is due to the fact that opposite edges were used for the edge loading of the first and second slabs.

The 2-point static load.—This special loading was applied in exactly the same manner as in the testing of the first slab. The loads were applied twice and the mean values of the measured deflections are shown in Figure 25. The strains produced by this loading at the strain gage positions shown in Figure 8 were too small to be measured with any accuracy except those at the mid-point of the stringers, where stresses of slightly less than 3,000 pounds per square inch were indicated.

There was no measurable spread between the T-bars at the center of the slab under the 2-point loading.

Application of the first impact.—In applying impact to the second slab the apparatus and procedure were the same as those used in the testing of the first slab. The average magnitude of the impact reaction in this first series of 1,000 drops was 27,300 pounds.

Following the application of this impact the testing machine was removed and the slab given a careful inspection. There was no visible effect of this impact on either the upper or lower surfaces of the slab. After this examination the static loading equipment was again placed on the slab.

Static loading after the first impact.—In accordance with the program, static loads of 5,000, 10,000, 15,000, and 20,000 pounds were next applied at the center point of the slab. Each load was applied twice and complete deflection and strain measurements were made for each load. The mean deflection data are shown in Figure 26 and the strain data, expressed as unit stresses, are shown in Figure 27.

If a comparison is made between these data and those given in the corresponding figures before impact was applied (figs. 22 and 23), it will be noted that after the impact the maximum deflection of the slab for a given load was slightly greater than before impact. For example, the deflection of the center point under the 20,000-pound static load was 0.073 inch before impact and 0.077 inch after impact, an increase of about 5 per cent. The comparison of the stress data will show that

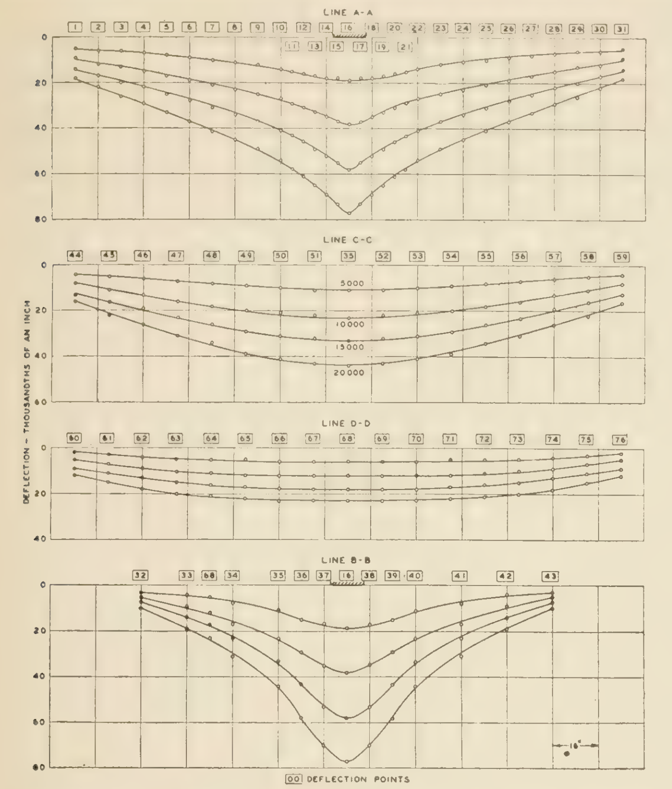

FIGURE 26.—DEFLECTIONS OF SECOND SLAB UNDER CENTER LOADING AFTER FIRST IMPACT

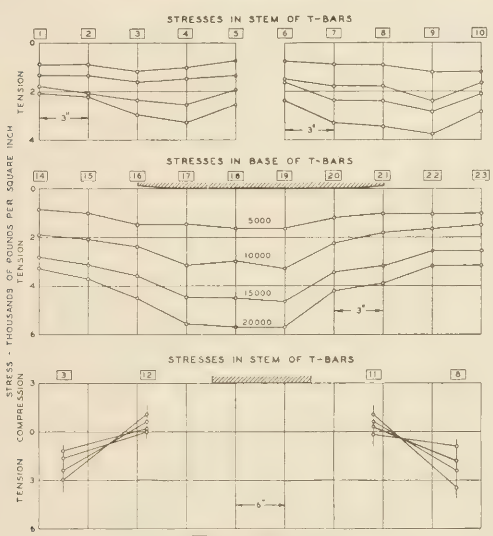

FIGURE 27.—STRESSES IN SECOND SLAB CAUSED BY STATIC LOADS AFTER FIRST IMPACT

there was some adjustment of the stresses in the T-bars near the load, but that this adjustment did not increase the maximum stress greatly for any of the loads. In the bases of the T-bars under the load there was apparently a small increase in the stresses due to the 15,000-pound load, while for the other three loads there was no increase. In the stems of the T-bars there was a small increase for most of the loads, but since the stress for the largest load was less than 4,000 pounds per square inch, this increase is of no particular importance except as another indication of a readjustment of the amount of load taken by the concrete and by the steel.

Application of the second impact.—In applying the second series of impact loads to the second slab the magnitude of the impact reaction was increased to an average value of 30,400 pounds by increasing slightly the height of free fall of the truck wheel of the impact machine. This corresponded to the value of 31,300 used in the second series on the first slab.

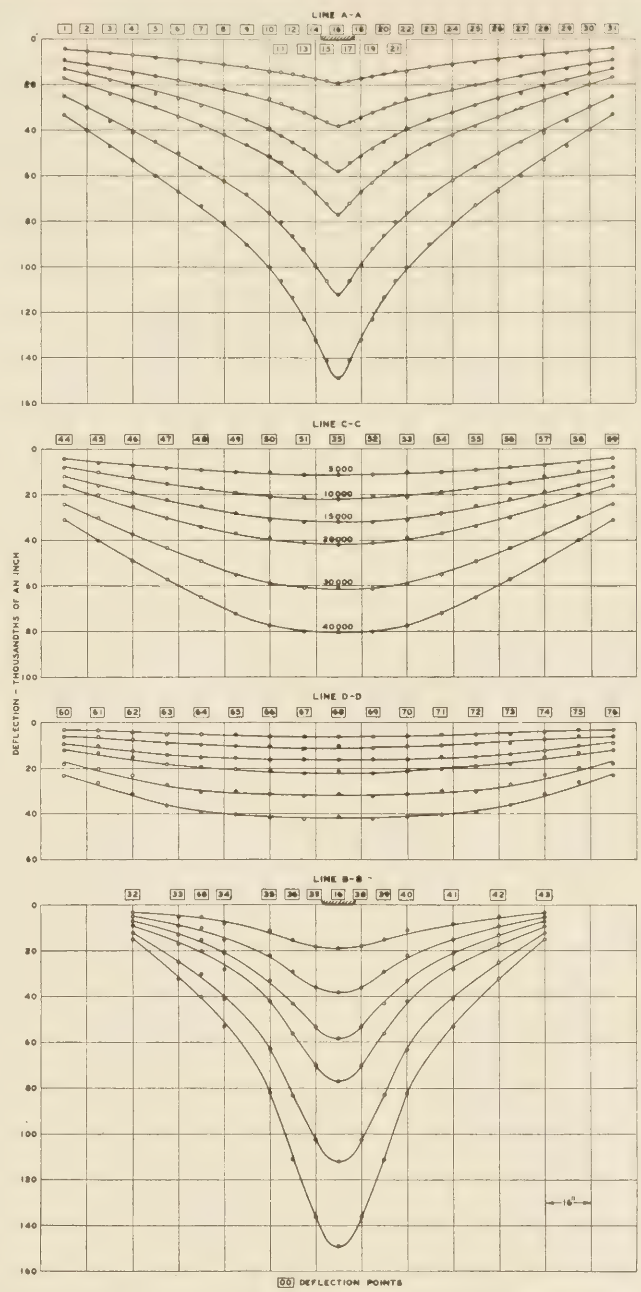

FIGURE 28.—DEFLECTIONS OF SECOND SLAB UNDER CENTER LOADING AFTER SECOND IMPACT

After the completion of 1,000 drops the impact machine was removed and the slab carefully inspected. There was no visible change in either surface of the slab.

Static loading after the second impact.—Static loads of 5,000, 10,000, 15,000, 20,000, 30,000, and 40,000 pounds were applied at the center of the slab. These loads were applied twice, complete deflection and strain measurements being made for each load. The mean deflection curves for these loadings are shown in Figure 28. The corresponding stress data for all of the strain gage positions are given in Table 3.

These data indicate that the application of the second impact did not produce any significant change

Table 3.—*Unit stresses in T-bars of second slab under center loading after second impact, in pounds per square inch*

TOP OF STEMS OF T-BARS

Load	Stress at strain gage point—											
	1	2	3	4	5	6	7	8	9	10	11	12
	Tension										Compression	
Pounds												
5,000	900	900	1, 200	1, 050	600	750	750	1, 050	600	1, 200	300	[1] 450
10,000	1, 350	1, 500	1, 950	1, 950	1, 500	1, 050	1, 350	1, 800	1, 500	1, 800	450	[1] 150
15,000	1, 950	2, 100	2, 850	2, 700	1, 950	1, 800	1, 800	2, 700	2, 100	2, 400	750	150
20,000	2, 850	2, 850	3, 750	3, 750	3, 000	2, 550	2, 700	3, 750	3, 150	3, 150	1, 050	300
30,000	4, 200	4, 650	5, 550	5, 550	4, 500	3, 750	4, 650	5, 550	4, 650	4, 800	1, 650	600
40,000	5, 700	6, 150	7, 500	7, 650	6, 300	5, 550	6, 450	7, 650	6, 900	6, 300	2, 400	1, 050

BOTTOM OF BASES OF T-BARS

Load	Stress at strain gage point—									
	14	15	16	17	18	19	20	21	22	23
	Tension									
Pounds										
5,000	1, 050	1, 200	1, 200	1, 650	1, 950	1, 350	1, 350	1, 200	1, 200	1, 200
10,000	1, 950	2, 100	2, 400	2, 700	2, 850	2, 400	2, 850	1, 800	1, 350	1, 650
15,000	2, 250	3, 000	3, 150	3, 750	4, 350	3, 750	3, 300	2, 850	2, 100	2, 100
20,000	3, 300	4, 050	4, 500	5, 250	5, 400	4, 950	4, 800	3, 600	2, 850	2, 850
30,000	5, 700	6, 750	7, 950	8, 700	9, 150	8, 550	7, 650	6, 000	5, 700	4, 950
40,000	7, 050	8, 400	9, 900	11, 400	12, 150	11, 100	9, 900	8, 100	7, 200	6, 750

[1] Tension.

in the behavior of the slab. The deflections were practically the same as those measured for corresponding loads after the application of the first impact and the load-deflection relation at the center of the slab is essentially a straight line up to and including the 40,000-pound load. The strains in the T-bars were practically unchanged for the four lower loads and the maximum stress indicated at any point was 12,150 pounds per square inch in the T-bar under the center of the 40,000-pound load.

Application of the third impact.—In this series the average impact reaction was 31,900 pounds, corresponding exactly to the value used in the application of the third series of impact loads to the first slab. After the completion of the series of 1,000 drops the slab was again carefully inspected but no visible effect of the impact was found.

Static loading after the third impact.—The final series of static tests comprised loads of 5,000, 10,000, 15,000, and 20,000 pounds applied at the center of the slab, complete deflection and strain measurements being made for each loading. The mean deflections are shown in Figure 29 and the stress data in Table 4.

A comparison of these data with those obtained under the same loads, before the third series of impact loadings was applied, shows that the third impact did not produce any significant change in the action of the slab.

Stresses in the stringers.—As in the case of the first slab, strain measurements were made at the mid-point of the two stringers nearest the center line of the slab. These strain data, expressed as unit stresses, are shown for all loads in Figure 30. It will be noted that there was practically no stress in the upper flanges and that even with the 40,000-pound load the stress in the lower flange did not exceed 7,000 pounds per square inch. The position of the neutral axis of these members is rather indefinite, because of the fact that the stresses in the upper flange of the stringers were so small as

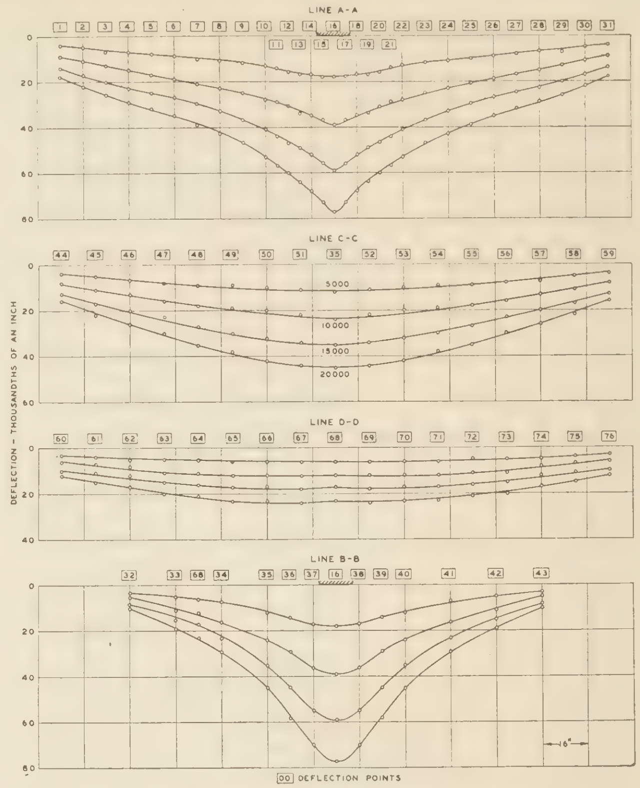

Figure 29.—Deflections of Second Slab Under Center Loading After Third Impact

Table 4.—*Unit stresses in T-bars of second slab under center loading after third impact, in pounds per square inch*

TOP OF STEMS OF T-BARS

Load	Stress at strain gage point—											
	1	2	3	4	5	6	7	8	9	10	11	12
	Tension										Compression	
Pounds												
5,000	900	600	900	600	750	750	750	750	900	750	[1] 150	[1] 300
10,000	1, 650	1, 350	1, 500	1, 500	1, 350	1, 200	1, 650	1, 650	1, 650	1, 200	0	0
15,000	2, 100	2, 250	2, 250	2, 400	2, 250	2, 250	2, 400	2, 550	2, 850	2, 250	450	300
20,000	2, 850	3, 150	3, 450	3, 300	2, 850	3, 150	3, 150	3, 600	3, 450	2, 700	450	150

BOTTOM OF BASES OF T-BARS

Load	Stress at strain gage point—									
	14	15	16	17	18	19	20	21	22	23
	Tension									
Pounds										
5,000	1, 050	900	1, 200	1, 200	1, 500	1, 350	900	1, 050	600	300
10,000	1, 650	1, 800	2, 100	2, 550	2, 700	2, 850	1, 800	1, 950	1, 650	1, 050
15,000	2, 550	2, 850	3, 450	4, 050	4, 050	3, 900	3, 000	2, 700	2, 550	1, 650
20,000	3, 600	3, 900	4, 650	5, 250	5, 100	5, 100	4, 050	3, 450	3, 300	2, 400

[1] Tension.

to be incapable of being measured accurately, but apparently it is near the lower surface of the slab.

Figure 31 shows the load-deflection and load-stress relations for the mid-point of the stringers and for the mid-point of the slab before and after the application of the impact. This figure is a companion graph to Figure 21, the values measured before impact being connected with a solid line and those observed after the three series of impact loadings being shown as plotted points. In these data there is an indication of the same increase in flexibility that was noted after impact was

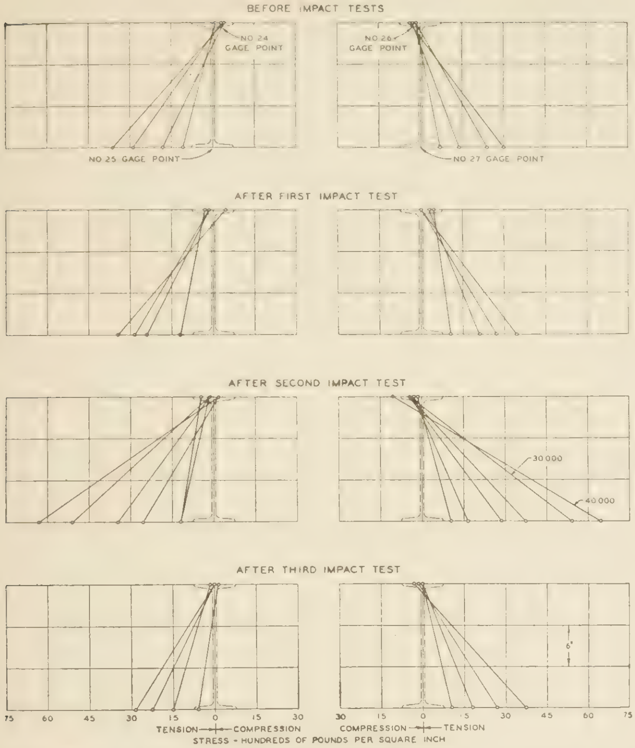

Figure 30.—Stresses in Second Slab at Midpoint of Stringers Under Static Loads of 5,000, 10,000, 15,000, and 20,000 Pounds

applied to the first slab but the magnitude of the increases both in deflection and in stress is very much less.

OTHER DATA OBTAINED

Deflection under impact.—At the conclusion of the scheduled tests on the second slab, advantage was taken of the opportunity to obtain some data on the relative deflections of the mid-point of the slab under impact and equivalent static loads.

The procedure used for measuring the slab deflection under impact was very simple. In the center of the base of the T-bar directly under the center of the load a short stylus point of hardened steel was attached. The stylus extended downward a short distance and the tip was bent over into a horizontal position. A substantial framework resting upon the earth beneath the slab supported, on edge, a small piece of plate glass, one side of which was coated with a smoke film. The smoked side of this glass plate was adjusted so that it just made contact with the horizontal stylus point. A deflection of the slab produced a short vertical trace in the smoke film. After each loading the glass plate was moved forward in guides to a new position, many records being made on one plate. The measurement of the length of the scribed lines was made with a precision comparator. With this instrument an accuracy of measurement of better than one thousandth of an inch was readily obtainable. This method of making deflection measurements was adopted because of the difficulty of accurate measurements of deflection under impact with any of the more usual forms of deflection-measuring equipment.

The data obtained from these measurements are shown in Figure 32 compared with the static load-deflection data obtained after the second series of impact loads were applied. This series was used because it included the 30,000 and 40,000 pound loads. It will be noted that the value of the impact pressures

Figure 31.—Comparison of Deflections and Stresses in Second Slab Before and After Impact

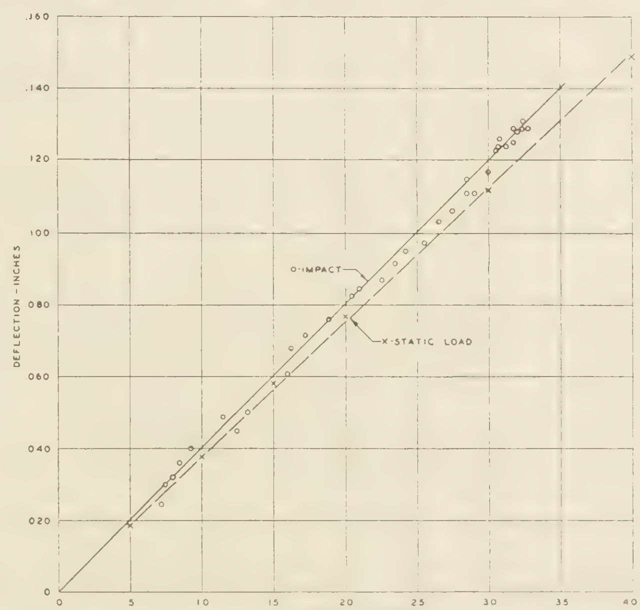

Figure 32.—Comparison of Deflections of Midpoint of Second Slab Under Impact and Static Loads

developed ranged from about 7,400 to about 33,000 pounds. The impact load values were those computed from the accelerometer measurements as previously described.

These data indicate little difference between the deflections of the mid-point of the slab under static loads and those caused by impact loads of the type imposed by the wheels of motor vehicles.

Dye test for separation of steel and concrete.—The increased flexibility of the slabs after the application of the first impact raised the question as to whether or not this was due to a breaking of the bond between the concrete and the stems of the T-bars in the vicinity of the area on which the impact was applied. A careful visual examination of this part of the first slab at the time of its demolition failed to show any evidence of such separation. After the testing of the second slab it was decided to stain the area subjected to impact with a penetrating dye so that, if any fissures had developed along the stems of the T-bars, the dye would penetrate and stain them, thus indicating their presence and extent when the slab was finally demolished. A similar area near the edge of the slab was also treated with the dye in order that the presence of fissures due to causes other than impact (as for example, shrinkage) would be revealed. The dye used was methyl violet base dissolved in alcohol and ponded over the area in question.

Some unreported experiments in the staining of fissures in concrete had shown this dye to be very satisfactory for the purpose, as it penetrates readily and produces an excellent stain.

When the slab was demolished it was found that there had been no penetration of the dye either along the sides of the T-bars or at any other point.

DISCUSSION OF THE DATA

In considering the data which have just been presented, it should be borne in mind that the deflection measurements are undoubtedly more reliable than the strain measurements because the magnitude of the displacement being measured was so much greater in the case of the deflections. For this reason personal errors, mechanical errors, and temperature errors in the measurements are generally a very small percentage of the total measurement in the deflection data. In the measurement of strains, however, such is not the case. The stresses in the steel were, in general, small and, even with the 10-inch gage length which was used, the total deformation was small, being of the general order of 0.0010 to 0.0030 inch. With such small displacements, the utmost care is necessary to keep the personal errors of strain gage manipulation and the temperature effects on the strain gage within reasonable limits.

All strain measurements were made by one operator in order to eliminate differences resulting from any personal equation. Check readings were made at periods when the air temperature agreed as nearly as possible with that which obtained during the initial readings, because it was found that, despite temperature corrections, changes of temperature of either the steel in the slab or of the gage itself could produce noticeable variations in the small strain values being measured. Corrections for temperature by means of a standard reference bar were made for all strain measurements.

It is believed that the data obtained with the strain gage in these tests indicate, with reasonable accuracy, the actual stresses which existed in the steel at the time of measurement.

Deflection of the slabs.—The general character of the deflection curves indicates that both of the slabs acted as stiff plates, giving a good distribution to the load.

In order to compare the behavior of the two slabs the following table has been prepared showing the deflections of the mid-point of the slab, of the mid-point of the stringer adjacent to the load, and of the floor beam at the stringer connections, for the 20,000-pound static load which was applied in the center of each slab both before and after the application of the first series of impact loadings.

TABLE 5.—*Gross deflections, in inches, under the 20,000-pound load applied at the center of the slab*

FIRST SLAB

	Mid-point of slab	Mid-point of stringers	Floor beam at stringer connection
Before impact	0.087	0.046	0.014
After first impact	.113	.057	.020
Increase	.026	.011	.006

SECOND SLAB

	Mid-point of slab	Mid-point of stringers	Floor beam at stringer connection
Before impact	0.073	0.043	0.015
After first impact	.077	.044	.016
Increase	.004	.001	.001

It is apparent that the second slab was a stiffer structure than the first slab. The deflection of the mid-point of the second slab with respect to the adjacent stringers was 0.030 inch as compared with that of 0.041 inch for the first slab. The deflection of the mid-point of the stringer with respect to its ends was 0.028 inch under the second slab and 0.032 inch under the first slab, indicating greater slab stiffness and consequent distribution of load on the stringer. These comparisons refer to deflections before impact.

The data show that the impact loadings served to increase the flexibility and thus to decrease the load-distributing ability of both slabs. In the first slab this change was very noticeable; in the second slab it was much less marked. If the deflection of the mid-point of the slab with respect to the adjacent stringers is compared it will be found that the deflection of the first slab increased from 0.041 inch to 0.056 inch (37 per cent), while for the second slab the increase was from 0.030 inch to 0.033 inch (10 per cent) after the application of the impact. Similarly, the increase in stringer deflection after the first impact was from 0.032 to 0.037 inch, or 16 per cent, for the first slab and zero for the second slab. This indicates that the impact caused a decrease in the distribution of the load on the stringers under the first slab but caused little or no change in this distribution for the second slab.

The deflection data show that after the first series of impacts subsequent loadings, either static or impact, caused little or no change in structural behavior, the action of both slabs being entirely elastic even for the 30,000 and 40,000 pound static loads.

While the causes for the differences which were observed in behavior of the two slabs have not been definitely determined, it is of interest to give some consideration to the various features of their structural design which might have been responsible for them.

The supporting structure—i. e., the walls, the floor beams, and the stringers—was the same for both slabs. As has been previously noted, the floor beams were bolted down on bearing plates and the stringers were rivet-connected to the floor beams. From these facts and such deflection data as were obtained for the floor beams and stringers, it seems improbable that any appreciable part of the differences observed was due to the substructure.

Among the structural elements of the slab proper there were several differences which may have had some influence. The base of the T-bar used in the steel mat of the second slab was appreciably thicker than that used in the first slab. (See fig. 6.) The effect of this would be to increase the stiffness of the slab and, to some extent, the stiffness of the stringers to which these members were welded. There were fewer welds between the bases of the T-bars and the stringers in the second slab than there were in the first. However, on reference to Figure 5, it will be seen that the abutting edges of the bases of the T-bars in the second slab were welded together at several places. There were no such welds in the first slab. The effect of these welds is to increase the continuity of the steel forming the bottom of the slab, and thus to increase the slab stiffness to some indeterminate extent. There is one other feature of the steel design of the slabs which probably had an important influence on the difference in stiffness noted, and that is the fact that in the second slab the upper edges of the stems of the T-bars were connected by rather heavy transverse members spaced at frequent intervals and welded in place. Since these members would be capable of receiving and carrying a considerable amount of compressive stress in the direction normal to the T-bars, their presence would give the steel mat a resistance to lateral compression quite independent of the concrete filling material.

The concrete in the two slabs was intended to be as nearly the same as possible. The proportions were the same, the aggregates were of the same quality, came from the same sources, and were of nearly the same gradations. Different cements were used for the two slabs, but the second cement was selected because its 28-day strength tests gave values almost identical with those obtained at the same age with the first cement. The first slab was placed when the air temperature was over 100° F. and the water in the mix was increased to a water-cement ratio of 0.86 before the concrete could be placed. It is probable that the concrete as it came from the mixer and was used in making the test cylinders had this water-cement ratio. It is improbable, however, that the concrete in the slab at the time the vibrating was completed had as much water in it as this water cement ratio would indicate. When the second slab was placed, the weather was much cooler and, with a water-cement ratio of 0.82, no particular difficulty was experienced in placing the concrete in the mats. Although the two sets of test cylinders cured in the damp room for 28 days showed a wide difference in concrete strength, it seems quite probable that the concrete in the two slabs at the time the load tests were made did not differ in its properties nearly as much as the strength test data from the 6 by 12 inch cylinders tested at 28 days would lead one to believe.

It is difficult to estimate just what part the concrete plays in the functioning of a slab of this type. At the time the concrete is placed it fills completely the rectangular trough between the T-bars. Subsequently, it dries out and shrinks in volume. The amount of the shrinkage which occurs in concrete during this period depends on many factors, including the kind and amount of cement and the amount of water present in the mixture. It is possible for a 3-inch prism of concrete to shrink sufficiently to draw completely away from the mold and the fact that very fine cracks were observed over the stems of the T-bars in both slabs, over the rectangular transverse members in the first slab and along the edges of the half-round transverse members in the second slab, suggested the possibility that such separation had occurred in these slabs.

These fine cracks were quite generally distributed over the entire upper surface of both slabs, were observed in many instances before any loads had been imposed, and underwent no apparent change during the application of the various loads. In a number of places in the discussion of the effect of loading reference is made to the presence or absence of structural cracks or cracks due to load and these should not be confused with the fine surface fissures due to shrinkage of the concrete which have just been mentioned.

Throughout the testing of the two slabs the behavior of the concrete was closely watched, yet no indication of a loosening of the concrete was observed at any time. When the static loads were applied at the center of the slab and the separation of the T-bars measured in the manner previously described, it was evident that even under the 5,000-pound load the measured separation at that point was sufficient to break such bond as may have existed before the static load was applied.

In spite of these evidences of separation between the concrete and the steel, it was found when the slabs were demolished that the concrete was adhering tightly to the steel; that when it was chipped off the surfaces of the steel members in contact were clean and free from rust; that the concrete showed no sign of rust stain; and, in the case of the second slab, that the dye which had been ponded on the upper surface of the slab had not penetrated the slab at any point. It must be concluded, therefore, that good bond between the steel and the concrete did exist and that this bond was maintained throughout the tests and until the time of the demolition of the slabs after an exposure of nearly a year.

In view of these facts it seems probable that the concrete acts as a filler receiving the load; that it serves to support the stems of the T-bars during bending along the transverse axis of the slab, enabling them to take full compression; that it takes some compression itself in this direction; and that during bending along the longitudinal axis of the slab it takes its proportionate share of the compression in the slab. It is indicated also that the steel surfaces which were covered by an adequate thickness of the concrete, such as the sides of the stems and the tops of the bases of the T-bars, were protected against rusting.

Stresses in the steel.—The tensile stresses at the center of the lower flange of the stringers were proportional to the deflection and were very moderate for all of the loadings. In the first slab test this stress was approximately 4,000 pounds per square inch for the 20,000-pound load and about 7,000 pounds per square inch for the 40,000-pound load. In the second slab test this stress amounted to approximately 3,500 pounds per square inch for the 20,000-pound loan and about 6,000 pounds for the 40,000-pound load.

The stresses in the stems of the T-bars at the positions of maximum negative moment, over the stringers, were moderate for all of the static loads. In the first slab these stresses were of the order of 6,000 pounds per square inch for the 20,000-pound load and 11,000 pounds per square inch for the 40,000-pound load. In the second slab the stresses for these same loads were of the order of 3,500 and 7,500 pounds per square inch, respectively.

The stresses in the bases of the T-bars directly beneath the load were the highest observed at any of the points where measurements were made. In the first slab this stress was about 7,500 pounds per square inch before impact and about 10,000 pounds per square inch after impact for the 20,000-pound load. For the 40,000-pound load it was about 20,000 pounds per square inch. In the second slab the 20,000-pound load caused a stress of about 6,000 pounds per square inch, the 40,000-pound load caused a stress of about 12,000 pounds per square inch, and the effect of the impact on the magnitude of the stress at this point was very slight. If the data concerning this stress in Figure 23 are compared with those in Figure 27, it will be seen that while the magnitude of the maximum stress was not changed by the application of the impact, the distribution of the stress among the T-bars under the load was changed to some extent.

To summarize, the stress data lead to the conclusion that the 20,000-pound static load applied at the center of the slab did not cause excessive steel stresses in either of the slabs tested.

General observations.—In the construction and testing of these two slabs there were several points observed which may be of general interest.

The mats, as received, were not flat and did not lie smoothly on the stringers. It was necessary to draw these mats down as they were welded in order that each T-bar might be in bearing on the stringers.

The welding of the mats to the stringers was accomplished without difficulty with an electric welding outfit. These field-made welds were found to be sound and intact when the slabs were demolished. The welds which attached the half rounds to the stems of the T-bars in the second slab were not uniformly good, a number of defective welds being found before the concrete was placed and during the demolition of the slab.

In the construction of both of the slabs the concrete was finished off practically flush with the upper surface of the steel mats, but the finishing left a thin layer of mortar over the top of the stems of the T-bars and over the transverse members in the mats. This layer was about one-sixteenth to one-eighth inch in thickness. During the period of exposure subsequent to testing, moisture penetrated this thin layer of mortar, rusted the steel, and forced off the mortar layer. At the time the slabs were demolished, it was found that in a good many places the steel which had been exposed in this way was deeply pitted by rust.

It seems advisable that some means be developed for protecting the steel in the upper surfaces of mats which are to be used for bridge floor slabs or other exposed structures. This might be accomplished through an increase in the thickness of the concrete cover over the steel or through the application of an impermeable wearing surface to the slab. Either of these would increase the weight of the floor somewhat but would insure the durability of the structure.

Concrete for the construction of floor slabs of this type should be designed for workability as well as strength. The mix which was used in these tests was somewhat deficient in workability and would have been improved by an increase in the proportion of fine aggregate in the mixture. The protection of the steel is a very important function of the concrete and can be assured only by dense, impermeable material. Vibratory methods of placing are of great assistance in securing thorough consolidation, but a properly proportioned concrete mixture is necessary. If the concrete is not workable, the tendency will be to overvibrate in an effort to secure consolidation, and this may be carried so far as actually to cause segregation and loss of the cement paste.

CONCLUSIONS

From the data which have been presented and from observations made during the investigation the following conclusions are drawn.

1. That both slabs acted as stiff plates giving a good distribution of the load to the supporting structure.
2. That both slabs were capable of supporting static wheel loads such as are normally found in present-day traffic, without excessive stress.
3. That the effect of the severe impact which was applied during the tests was to increase the flexibility of the structure and to decrease the distribution of the load. This effect was more marked in the first slab than it was in the case of the second slab.
4. That there was nothing in the behavior of the slabs under static loads subsequent to impact to indicate that either slab had been damaged structurally by the impact.
5. That there was no loosening of the concrete by the impact.
6. That, for durability in exposed locations, the steel in the upper surface of the slabs should have more protection from moisture than was provided in the slabs tested.

BITUMINOUS CONCRETE ON CONNECTICUT AVENUE EXPERIMENTAL ROAD

The photographs on the cover of Public Roads this month depict a portion of the Connecticut Avenue experimental road which was constructed in the fall of 1912. The back cover shows the Portland cement concrete base ready for the application of the bituminous concrete surface and the appearance of the pavement one year after completion. The view on the front cover, taken at the same location in August, 1932, reveals the present condition of the surface.

This section, known as experiment No. 2 (north of Bradley Lane), is an asphaltic concrete proportioned in accordance with the District of Columbia specifications and laid 2 inches thick on a foundation of 1:3:7 gravel concrete. The experiment was divided into two sections in which limestone and trap-rock screenings, respectively, were used. Limestone dust was used as filler and the binder was a fluxed native asphalt.

The construction cost was 195.65 cents per square yard. The section has given very good service for 20 years under an increasing burden of traffic, which in 1931 reached an average daily density of more than 3,000 vehicles. Wear and depressions have developed near the gutter, and have been repaired with cold-patch mixtures of tar and stone chips. The total maintenance costs during the 20-year period have been 20.38 cents per square yard.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

AUGUST 31, 1932

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION						APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal-aid allotted	Percentage completed	MILEAGE			Estimated total cost	Federal-aid allotted	MILEAGE				
					Initial	Stage[1]	Total			Initial	Stage[1]	Total		
		$	$					$	$				$	
Alabama	2,405.2	1,638,442.00	797,324.95	97	62.8		62.8	58,725.70	29,362.85	0.1		0.1	5,441,650.11	Alabama
Arizona	1,263.0	2,088,285.46	1,439,484.25	74	73.3	61.7	135.0	272,862.12	185,957.62	11.4	11.2	22.6	622,993.79	Arizona
Arkansas	1,967.1	2,262,708.94	1,033,985.95	85	64.5	38.4	102.9	362,032.53	181,016.26	17.3		17.3	2,183,603.25	Arkansas
California	2,381.0	9,361,726.01	4,457,601.41	61	212.9	41.3	254.2	456,367.55	27,613.68	11.2		11.2	149,658.64	California
Colorado	1,635.7	3,788,907.63	2,005,478.03	83	151.4	45.7	197.1	110,645.42	57,288.90		12.5	12.5	1,905,004.32	Colorado
Connecticut	287.0	4,071,872.74	1,479,495.06	76	24.9		24.9	739,583.09	350,534.32	10.9		10.9	316,492.14	Connecticut
Delaware	368.7	713,766.00	356,883.00	82	22.9		22.9						167,140.82	Delaware
Florida	660.2	3,363,074.03	1,543,777.03	77	83.5		83.5	281,309.68	140,654.83	14.0		14.0	2,554,760.29	Florida
Georgia	3,196.8	5,350,186.08	2,469,490.42	69	103.7	96.6	200.3	2,358,686.34	1,050,904.76	52.4	57.1	109.5	281,185.36	Georgia
Idaho	1,321.2	2,796,422.18	1,649,205.10	83	120.9	245.9	366.8	932,602.72	443,424.06	53.1	78.9	132.0	316,199.72	Idaho
Illinois	2,747.3	24,693,217.74	11,553,396.48	81	765.0	37.3	802.3	1,269,133.53	492,831.19	32.4	5.3	37.7	1,282,514.10	Illinois
Indiana	1,878.8	9,798,463.34	4,861,495.34	73	360.3	20.3	380.6	673,152.52	336,572.23	31.9		31.9	330,586.66	Indiana
Iowa	3,321.7	5,596,084.04	2,623,065.52	56	279.8	65.8	345.6	2,192,863.61	171,393.99	92.2	32.3	124.5	17,616.03	Iowa
Kansas	3,667.4	5,011,486.05	2,442,297.86	68	246.4	112.1	358.5	1,438,881.65	696,511.49	52.8	190.3	243.1	479,029.36	Kansas
Kentucky	1,864.1	3,266,669.30	1,292,885.74	64	150.1	65.0	215.1	981,468.60	442,011.36	32.5	40.3	72.8	849,990.74	Kentucky
Louisiana	1,568.4	7,007,686.01	3,279,075.14	51	86.1	9.9	96.0						900,320.80	Louisiana
Maine	743.7	3,049,320.48	1,286,264.65	64	94.0		94.0	339,792.93	99,336.57	9.4		9.4	131,674.54	Maine
Maryland	794.5	1,756,875.28	758,362.78	37	66.3	4.7	71.0	145,106.45	51,453.72	5.4	1.5	6.9	23,131.64	Maryland
Massachusetts	821.6	6,062,373.37	2,458,736.42	84	62.7	6.6	69.3	1,092,212.22	262,592.11	11.8		11.8	242,294.74	Massachusetts
Michigan	2,119.7	8,526,956.39	3,919,188.29	73	343.5	36.8	380.3	1,189,776.40	493,277.50	32.1	38.0	70.1	1,718,969.86	Michigan
Minnesota	3,935.9	12,021,979.80	3,899,301.38	78	215.9	317.0	532.9	3,849,615.44	71,000.00	77.9	110.7	188.6	9,000.00	Minnesota
Mississippi	1,809.3	3,980,188.06	1,950,466.97	87	187.3	68.6	255.9	269,411.54	134,705.76	25.9		25.9	5,038,133.67	Mississippi
Missouri	2,935.7	6,824,609.23	2,949,242.60	68	178.0	97.1	275.1	361,486.67	173,742.54	18.6	2.9	21.5	600,971.81	Missouri
Montana	2,733.2	3,626,063.17	2,024,078.26	79	320.9	62.9	383.8	2,362,324.82	1,194,199.75	173.7	212.9	386.6	2,073,920.45	Montana
Nebraska	4,235.0	3,681,387.37	1,799,950.59	54	141.2	82.2	223.4	896,296.08	415,446.03	13.7	64.0	77.7	1,799,883.40	Nebraska
Nevada	1,305.2	1,735,515.14	1,468,486.20	65	37.1	150.0	187.1	123,288.21	109,469.42		35.0	35.0	54,712.94	Nevada
New Hampshire	424.6	551,782.15	258,211.90	45	21.0		21.0	73,997.91	29,599.16	2.8		2.8	361,439.50	New Hampshire
New Jersey	623.6	5,446,567.60	2,120,645.78	44	28.2	.5	28.7	135,582.41	33,895.60	2.3		2.3	595,474.00	New Jersey
New Mexico	2,242.7	1,860,648.39	1,296,917.91	74	84.8	25.2	110.0	437,617.69	248,140.13	11.7	28.7	40.4	963,522.93	New Mexico
New York	3,280.2	16,050,960.00	7,470,955.00	59	420.9		420.9	7,560,950.00	1,849,130.00	143.8	2.2	146.0	1,457,362.65	New York
North Carolina	2,222.3	1,198,731.16	596,538.91	82	67.9	9.0	76.9	1,003,029.10	500,697.17	114.9	17.2	132.1	3,349,069.60	North Carolina
North Dakota	5,053.5	4,050,618.86	1,941,159.65	51	456.7	383.4	840.1	1,784,693.34	703,968.66	109.2	394.8	504.0	434,922.78	North Dakota
Ohio	2,859.7	9,321,042.45	3,131,000.05	62	191.2	61.2	252.4	5,105,956.23	1,345,732.45	122.6	36.5	159.1	787,110.07	Ohio
Oklahoma	2,334.7	3,453,586.37	1,710,268.34	76	183.6	36.7	220.3	1,641,623.19	844,147.14	82.1	17.6	99.7	873,326.65	Oklahoma
Oregon	1,531.2	3,530,472.22	1,945,315.51	67	140.8	69.3	210.1	413,400.04	175,162.65	14.3	27.9	42.2	910,135.52	Oregon
Pennsylvania	3,035.0	8,671,792.49	4,210,561.60	54	283.9		283.9	4,539,861.51	1,559,801.24	134.1		134.1	658,754.45	Pennsylvania
Rhode Island	258.0	722,705.49	357,240.59	82	16.4		16.4						326,325.34	Rhode Island
South Carolina	2,049.3	2,366,835.31	1,065,169.32	99	60.4	41.4	101.8						746,947.25	South Carolina
South Dakota	4,054.7	3,894,685.47	2,115,357.51	58	285.4	188.0	473.4	998,632.24	284,520.46	76.2	93.4	169.6	379,470.73	South Dakota
Tennessee	1,690.9	1,368,046.88	680,569.31	49	41.7	21.8	63.5	923,890.77	461,945.34	36.3		36.3	2,721,634.29	Tennessee
Texas	7,765.9	14,340,432.95	6,587,011.01	70	681.6	294.4	976.0	3,813,575.10	1,734,665.44	170.5	216.5	387.0	2,404,503.19	Texas
Utah	1,137.9	1,358,969.87	990,745.08	72	84.1	112.6	196.7	585,916.45	284,143.43	46.2	25.6	71.8	523,790.08	Utah
Vermont	339.5	1,369,875.57	611,039.17	56	45.2		45.2							Vermont
Virginia	1,910.6	2,088,351.39	1,008,598.75	80	112.2	18.0	130.2	1,375,468.12	624,322.93	72.9	9.7	82.6	1,455,742.22	Virginia
Washington	1,222.0	2,977,682.18	1,417,494.35	79	99.8	13.2	113.0	772,237.61	232,300.00	17.4	6.1	23.5	743,126.17	Washington
West Virginia	894.0	1,939,105.52	819,108.45	81	47.7	10.8	58.5	1,246,166.98	473,666.56	46.6		46.6	607,091.95	West Virginia
Wisconsin	2,580.0	8,681,376.92	3,505,398.32	68	231.1	89.4	320.5	771,083.85	134,600.00	17.9	24.4	42.3	279,738.52	Wisconsin
Wyoming	1,952.3	3,413,913.88	2,127,834.84	82	179.4	215.1	394.5	174,698.46	114,631.32	22.9		22.9	18,577.01	Wyoming
Hawaii	74.8	1,442,767.53	920,136.34	43	40.7	1.7	42.4	38,476.35	38,476.35		1.9	1.9	1,335,978.89	Hawaii
TOTALS	101,504.8	242,175,216.49	112,686,297.11	69	8,260.1	3,257.6	11,517.7	56,154,483.17	19,284,846.97	2,025.4	1,795.4	3,820.8	51,425,482.97	TOTALS
Construction completed		168,307,000	77,991,000											
Balance uncompleted		73,868,000	34,695,000											

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

CONNECTICUT AVENUE EXPERIMENTAL ROAD, CHEVY CHASE, MD., NOVEMBER 11, 1912
PORTLAND CEMENT CONCRETE BASE READY FOR CONSTRUCTION OF BITUMINOUS CONCRETE SURFACE

CONDITION OF BITUMINOUS CONCRETE SURFACE AFTER ONE YEAR OF SERVICE, DECEMBER, 1913

U. S. GOVERNMENT PRINTING OFFICE: 1932